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Cross references may be made (1) to allied main headings or (2) to subheadings of main headings. In the latter case, subheadings are indicated in cross references by inclosure in parentheses. Headings and subheadings consisting of several words each may be indicated in cross references by initial words only.

Abbreviations used in this index are the following:

fn.—footnote.

r.—reference, referred to.

r. to p.—reference(s) to publication(s) by or concerning.

U. S. D. A.—United States Department of Agriculture.

U. S. D. C.—United States Department of Commerce.

U. S. S. R.—Union of Soviet Socialist Republics.

v.—versus.

Names of headings repeated in subheadings.

Names of States.

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A ROAD CONDITION AND SUBGRADE SURVEY PARTY IN THE FIELD

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SUBGRADE STUDIES OF THE BUREAU OF PUBLIC ROADS¹

Reported by C. A. HOGENTOGLER, Highway Engineer; I. B. MULLIS, Associate Engineer of Tests; and A. C. BENKELMAN, Assistant Research Specialist, all of the Division of Tests, United States Bureau of Public Roads

AT THE present time very little information concerning the influence of the subgrade on pavement behavior is assembled in such form as to be utilized by the highway engineer. Until recently, research on subgrades has been devoted largely to laboratory tests and gratifying progress has been made. Practical field and laboratory tests have been developed for measuring volumetric change and supporting value, assuming that these are primary influencing characteristics of subgrade soils. In accordance with this assumption soils with high moisture equivalents and shrinkage values would be classed as poor subgrade and those having comparatively low values in these tests would be classed as good subgrade.

DATA RELATING SUBGRADE TO PAVEMENT CONDITION NEEDED

However, our knowledge of the relation between pavement condition and the subgrade is meager. Little is known of the relative life or condition of pavements laid on good and questionable subgrades. Do increased cracking, breaking, rutting, raveling, and unevenness accompany bad subgrades? How are maintenance and operating costs influenced by subgrade condition? What additional initial expenditure on account of poor subgrade would be warranted by the difference in renewal, maintenance, and operating costs? Shall this additional expenditure be made on the subgrade, the pavement, or both? Can less thickness of pavement or a different type be used when the moisture equivalent is 15 instead of 30, and what test values bound the limits of good, intermediate, and poor subgrade?

Examination of existing reports reveals but few references to soil-test results translated into terms of road condition. The most specific of these is found in an article by A. C. Rose.² As a result of observations in the Pacific Northwest he states that when the lineal shrinkage of a soil exceeded 5 per cent, the pavement showed degrees of deterioration corresponding to the amount by which the shrinkage exceeded this value and, conversely, that when the lineal shrinkage was less than 5 per cent the pavements were found to be in relatively good condition.

The average highway engineer, without knowledge of any soil tests, can determine by inspection the location of subgrade of questionable character with a fair degree of precision. Remedial measures, however, based upon the qualitative analysis of the subgrades are yet to be determined.

The Highway Research Board has found that the use of steel reinforcement in concrete slabs is one method of compensating for bad subgrade, but no correlation of the amount, type, or design of the reinforcement with laboratory soil tests has been made. Some engineers believe the remedy lies in the use of subbase,

but definite information regarding the type and thickness is lacking. The relative economy of subbase and special pavement designs for specific cases must be determined.

The aim of the subgrade studies of the Bureau of Public Roads is to furnish the highway engineer with definite information on these questions. Its objects as submitted to the committee on tests of the American Association of State Highway Officials are to determine:

1. The practicability of varying the type or design of road surface to suit subgrade conditions.
2. The efficiency of various types of drainage, subgrade treatment and subbase.
3. The relative economy of subgrade correction (treatment, drainage or subbase) and change in pavement design (type, thickness or features) as compensation for varying subgrade conditions.

EXISTING INFORMATION AND METHODS INSUFFICIENT AS BASIS FOR CONCLUSIONS

A proposed method of attack was to correlate studies of roads in service with United States Bureau of Soils data, laboratory tests on subgrade soils, and cost records. The procedure required a classification of soils with regard to their physical properties, but laboratory tests then in use did not yield results which were adapted to a simple classification. To aid in solving these problems Dr. Charles Terzaghi, of the Massachusetts Institute of Technology, was retained by the bureau as research consultant. He suggested a preliminary grouping of soils based on region and simplified soil tests, and a final classification based on refined tests of representative samples of the preliminary groups.³ He summed up the value of laboratory and field work as follows:

Laboratory tests identify the raw materials of soils and are of value only for soil classification. Tests on specially prepared specimens in the laboratory do not necessarily indicate the physical properties of cracked, crumbled, or compacted soils in the field. Nor can laboratory tests, alone, indicate the efficiency of a soil when used as subgrade.

Only by field surveys can the influence of subgrade on pavements be determined. Whether or not a certain type or design of pavement is adequate for a given subgrade condition can be determined only by a study of the specific pavement laid on the given subgrade.

This statement indicates the importance of studies of roads in service, but such studies are a new departure in highway research and standardized methods of procedure did not exist. A tentative working plan for making road surveys has been developed by the bureau and has been tried on several roads to determine its workability. The complete outline with a description and discussion of the road-survey procedure follows.

¹ A report to the committee on structural design of highways, National Research Council, December, 1926.

² Field Methods Used in Subgrade Surveys, PUBLIC ROADS, vol. 6, No. 5, July, 1925.

³ Simplified Soil Tests for Subgrades and Their Physical Significance by Dr. Charles Terzaghi, PUBLIC ROADS, vol. 7, No. 8, October, 1926.

PROCEDURE FOR SUBGRADE STUDIES OUTLINED

I. Studies of roads in service, including experimental roads.

A. Field surveys.

1. Selection of pavements for condition surveys with regard to subgrade variables as follows:
 - a. Variation in soil type (as shown by county maps of the United States Bureau of Soils) under the same road or under roads comparable as to age, traffic, climate, and design.
 - b. Special features (drainage, subgrade treatment or subbase) under only part of the same road or under different roads comparable as to age, traffic, and design.
 - c. Same soil type or subgrade features under roads different in design or type but comparable as to age, traffic, or climatic influences.
2. Detailed procedure for pavement and subgrade surveys to include procurement of—
 - a. Record of pavement condition in detail (cracking, breaking, rutting, settlement, disintegration, unevenness, replacements, etc.).
 - b. Record of cut or fill, grade, excessive moisture, etc., and soil series, type, and layer. Soil inspection to be made by an experienced soil surveyor.
 - c. Samples of subgrade soil for laboratory test.
 - d. Exact limits of special features (drainage, treatment, subbases) and soil types.
 - e. Samples of road surface and also original soils for sand-clay and similar roads.

B. Analysis of pavement condition records including those now available and those to be secured.

1. Classification of pavements with regard to climate, age, traffic, type, and design.
2. Grouping of pavements with regard to subgrade soil classification using the United States Bureau of Soils terminology.
3. Grouping of pavements with regard to subgrade soil classification based on laboratory tests.
4. Grouping of pavements with regard to artificial drainage, subgrade treatment, and subbase.
5. Tabulation of pavement characteristics in cuts, on original ground surface and on fills, in regard to the four preceding classifications.

II. Subgrade laboratory procedure.

A. Routine tests and classification.

1. Determination of series, type, and general characteristics of the soil by reference to maps and reports of the United States Bureau of Soils.
2. Preliminary classification.
 - a. Color determinations (optional) on both wet and dry pats for that fraction of the soil which passes the 0.5-millimeter screen.
 - b. Sedimentation (optional). Settlement of soil after being thoroughly mixed with water in a 50-centimeter graduate. For each soil type samples are to be grouped according to color and sedimentation characteristics. Samples will be selected from these groups for routine testing in order to reduce laboratory testing to a reasonable amount.
 - c. Revised mechanical analysis. Determination of fractions of soils passing the 0.5, 2, and 5 millimeter screens. Coarse material classified as rounded, angular, etc.
 - d. Lower liquid limit for 0.5-millimeter fraction.⁴
 - e. Lower plastic limit for 0.5-millimeter fraction.
 - f. Shrinkage limit for 0.5-millimeter and 2-millimeter fractions.
 - g. Shrinkage rate for 0.5-millimeter and 2-millimeter fractions.
 - h. Slaking value for 0.5-millimeter fraction.⁵
 - i. Moisture equivalent for 2-millimeter fraction.
 - k. Volumetric change computed at moisture equivalent percentage from shrinkage limit and ratio on 2-millimeter fraction.
3. Final classification. Terzaghi swelling test and bearing power determination on representative soil samples only.
 - a. Modulus of compression.
 - b. Modulus of expansion.
 - c. Coefficient of permeability.

II. Subgrade laboratory procedure—Continued.

A. Routine tests and classification—Continued.

3. Final classification—Continued.

- d. Compressive strength of oven-dried specimens.
- e. Compressive strength of specimens compacted from lower liquid limit to pressure of 3 kilograms per square centimeter.

4. Additional tests.

- a. Special tests as desired or requested.

B. Research in soil tests. Additional work as follows:

1. Lower liquid limit.
 - a. Determination of air content of representative samples at lower liquid limit.
 - b. Comparison of results of standard lower liquid limit test with those obtained with samples on which a vacuum is produced.
 - c. Development of an automatic device for making the lower liquid limit test.
2. Lower plastic limit.
 - a. Influence of final thread diameter on results.
 - b. Comparison of results when threads are rolled on paper (different kinds) and glass.
3. Slaking value.
 - a. Influence of temperature.
 - b. Influence of size and form of specimen.
 - c. Development of standard test.
4. Volumetric change.
 - a. Study of errors of observation connected with the mercury and paraffin method for determining the volume of soils.
 - b. Development of a method for determining the volume of soils in a natural or undisturbed condition.
5. Specific gravity determinations.
 - a. Influence on result when cold water, boiling water, and different organic liquids are used.
6. Preparation of test specimen.
 - a. Development of a method for preparing soil powder and water, excluding air.
7. Swelling tests.
 - a. Influence of air content in soil samples.
8. Bearing power tests.
 - a. Influence of size of sample.
 - b. Influence of size of bearing block.
 - c. Improvement of apparatus to allow release as well as application of load.

C. Soil classification.

1. Preliminary classification of all routine soil samples.
 - a. Grouping of soils by regions with regard to geological and climatic conditions.
 - b. Classification of each regional group with regard to lower liquid limit as follows:

Class	O	Lower liquid limit
	I	10.0 to 14.2.
	II	14.2 to 20.0.
	III	20.0 to 28.4.
	IV	28.4 to 40.0.
	V	40.0 to 56.8.
	VI	56.8 to 80.0.
		80.0 to 113.6.

- c. Subdivision of each lower liquid limit class with regard to plasticity index as follows:

Class	a	Plasticity index
	b	Less than 1.
	c	1 to 7.
	d	7 to 14.
	e	14 to 21.
	f	21 to 28.
	g	28 to 35.
	h	35 to 42.
		42 to 49.

- d. Further subdivision, should some other simple test, such as dye adsorption, be found to identify soil characteristics other than those disclosed by the Atterberg plasticity tests.
- e. Identification of each soil type according to the United States Bureau of Soils system and terminology. Thus, test results and other soil data published by the Bureau of Soils can be utilized.
2. Final classification according to the Terzaghi swelling tests of representative samples from each of the preliminary groups.
 - a. Modulus of compression.
 - b. Modulus of expansion.
 - c. Coefficient of permeability.

⁴ Adaptation of Atterberg Plasticity Test for Subgrade Soils, by A. M. Wintermyer, PUBLIC ROADS, vol. 7, No. 6, August, 1926.

⁵ Slaking Value Test, a paper presented by Prof. F. H. Eno at the sixth annual meeting of the Highway Research Board, National Research Council, December, 1926.

III. Additional investigation.

- A. Determination of basis of correction for influence of coarse materials in subgrade. Since laboratory tests are confined to that fraction of soil passing the 0.5 millimeter screen, the characteristics of an entire subgrade soil sample can not be learned until a basis for correction for both round and angular grains has been established for all tests.
- B. Determination of relative efficiency of various types and designs of pavements with different loads and subgrade supports. Soil pressure cells to be installed under selected pavements at the time of construction. From each location, subgrade soil samples will be obtained and sent to the laboratory for identification and classification. At subsequent periods observations will be made as follows:
 1. On concrete pavements the subgrade support to be determined with pressure cells and the fiber deformation determined with graphic strain gauges for wheel loads of different magnitude.
 2. Macadam and bituminous pavements.
 - a. Determination of subgrade support and local pavement deflection for wheel loads of different magnitude.
 - b. Supplemental investigations to determine the magnitude of repeated deflections which macadam and bituminous pavements can safely withstand.
- C. Evaporation, freezing, and other tests relating to the physics of the subgrade.
- D. Detailed investigation of specific road failures to determine the conditions which cause failures.
- E. Experimental sections. Whenever the opportunity occurs, sections to be constructed to test corrective measures for questionable subgrade conditions. Future observations will be made on these sections to determine their efficiency.

IV. Digest of preceding investigations.

- V. Correlation of road studies (condition surveys) with soil classification (laboratory studies).
 - A. Influence of type of soil on pavement behavior.
 - B. Influence of soil characteristics (variations within types) on pavement behavior.
 - C. Determination of type and design of pavement for a given subgrade condition.
 - D. Determination of efficiency of various types of drainage, subgrade treatment, and subbase.
- VI. Correlation of road studies and subgrade investigations with office records.
 - A. Influence of road condition on maintenance costs.
 - B. Influence of type of soil on maintenance costs and life of pavement.
 - C. Influence of soil characteristics on maintenance costs and life of pavement.
 - D. Relative cost of compensating for variation in subgrade condition with:
 1. Change in type or design of pavement.
 2. Drainage, subgrade treatment or subbase.
 3. Change in both surface and subgrade.

It will not be possible to start all of the proposed investigations immediately. All those having a bearing on the subject have been listed so that a definite working plan for evaluating subgrade variables can be presented. Every effort is being made to start the road condition surveys as soon as possible.

ROAD CONDITION SURVEYS TO BE MADE

The idea of assembling information on the relative influence of the various factors affecting pavement behavior is not new. Extensive surveys covering the entire State highway system have been carried on as part of the routine work in Pennsylvania, North Carolina, and Washington, while a very complete survey has been made in California.⁶ The Highway Research Board has also assembled considerable valuable information in connection with its study of the economic value of steel reinforcement in concrete road surfaces.⁷

The Bureau of Public Roads and others interested in highway research have advocated the use of condition surveys in connection with subgrade studies.⁸ As part of the discussion on structural design of pavements at the 1925 meeting of the Highway Research Board, Clifford Older⁹ directed attention to the need of a more comprehensive study when he suggested a condition survey with the view of correlating soil classification and pavement condition. The Michigan State highway department is now engaged in an investigation of this kind.¹⁰

To make a survey of the many thousands of miles of roads now in existence would be too great a task. Accordingly, roads must be selected which will furnish the maximum amount of information with the smallest mileage of inspections.

Study of the effect of subgrade is the primary object, but it is appreciated that the data will yield information on every variable affecting pavement behavior. Before the influence of subgrade can be determined, the effect of other variables must be understood and this can be accomplished by confining observations to roads with but one variable. Variation in the road surface itself presents a difficulty but it is expected that valuable information will be derived from such experiments as the Virginia demonstration road,¹¹ in which the influence of climatic, construction, and curing variables on various designs are being determined.

If the resulting information is to be conclusive, it is essential that the roads inspected shall have acquired sufficient age or shall have been subjected to enough traffic to have developed defects, and it will also be necessary to pay particular attention to the modern types of construction, devoting only a limited amount of study to the obsolete types. It seems advisable to select only those roads with a recorded history of design and construction, as the analysis of any condition survey is a complex problem even when all data are available. In order to simplify and expedite the work, the field surveys will be of three separate classes.

Class 1 surveys will deal with roads which have failed only in part of their length. An effort will be made to determine if the failures were caused by soil characteristics which can be foreseen in future road construction. These studies will be made in detail from the viewpoint of the geologist, the soil scientist, and the highway engineer.

Class 2 surveys will deal with roads where corrective measures have been used for offsetting undesirable subgrade conditions on part of the length. These corrective measures may consist of a change in type or design of pavement, subgrade treatment, subbase, or special artificial drainage. The object of the surveys will be to determine the relative efficiency of the various corrective measures now in use. Class 1 and class 2 surveys will be carried on by separate parties.

Class 3 surveys will deal with roads laid on different types of soil. The object will be to determine the relative frequency with which pavement defects occur on different kinds of soil. Roads will be listed with regard to age, type, design, traffic, and climate. The

⁸ Practical Field Tests for Subgrade Soils, by A. C. Rose, PUBLIC ROADS, vol. 5, No. 6, August, 1924.

⁹ Proceedings of the Fifth Annual Meeting of the Highway Research Board, Dec. 3-4, 1925, Part 1, pp. 129-130.

¹⁰ Survey of Soils and Pavement Condition in Progress in Michigan, by V. R. Burton, PUBLIC ROADS, vol. 7, No. 4, June, 1926.

¹¹ Virginia Building Demonstration Road, PUBLIC ROADS, vol. 7, No. 6, August, 1926.

⁶ Report of a Study of the California Highway System by the U. S. Bureau of Public Roads, 1920.

⁷ Report of Investigation of the Economic Value of Reinforcement in Concrete Roads, by C. A. Hogentogler: Part 2, Proceedings of the Fifth Annual Meeting of the Highway Research Board, December, 1925.

United States Bureau of Soils maps will then be consulted and the final list for field study will embrace those roads which include the greatest number of soil variables. For study of maintenance costs as influenced by the subgrade various projects will be compared, each of the same age, type, and design and as nearly as possible subject to the same traffic and climate, but with different subgrade soils.

SURVEYS TO YIELD DETAILED INFORMATION ON ALL FACTORS

The condition survey records will show all pavement defects. On concrete roads there will be recorded all controlled and uncontrolled cracking, corner breaks, general breaking and surface checking, scaling, and

soil at points sufficiently close together to identify all existing soil types. Samples of the soil for laboratory analysis will be secured at the time of the inspection from both the subgrade and soil layers. The condition of the soil with respect to compaction and degree of saturation will also be noted as well as any changes in gradation, color, and character. Data will be recorded on water seepage in cuts and free water where its presence is indicated. It is intended to have a special soil survey made by a soil scientist where surveys have not been made by the Bureau of Soils and where any difficulty in determining the soil types is encountered.

STANDARD SYSTEM OF RECORDING TO BE USED

Figure 1 illustrates the symbols to be used and manner of recording the slab condition with respect to joints, cracks, corner breaks, general failures, raveling, and width of cracks and joints. Conventional symbols for indicating the classification of the corner breaks, general failures, and culverts, bridges, and cross-roads are also shown. The elevation of the pavement with reference to the adjacent terrain is shown continuously by a profile system. Other features are to be indicated as shown.

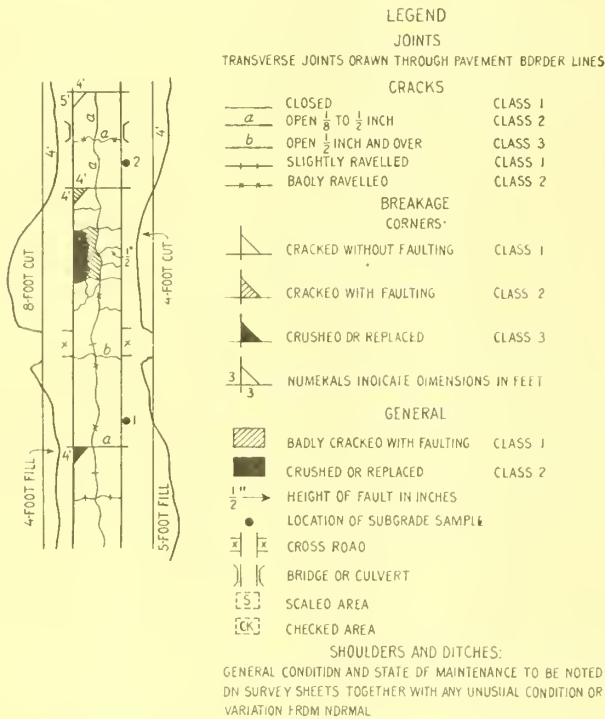


FIG. 1.—METHOD OF RECORDING AND SYMBOLS TO BE USED IN PAVEMENT CONDITION AND SUBGRADE SURVEYS

raveling. The width of the cracks and the degree of raveling, if any, are also important. A slab may be cracked badly and yet present a smooth, uniform surface; while another may be cracked to the same extent but be so uneven as to make early replacement necessary. Differences of this kind are to be noted in the reports. Information as to differences in condition is as important as the record of the breaks. All concrete pavements show surface defects; it is the magnitude and degree of these phenomena which merit attention in the survey.

The condition of the shoulders and ditches and the elevation of the shoulders of the roadway in relation to the pavement are points of particular interest both as to the condition of the surfacing and the subgrade. Information upon the general topography of the land adjoining the road and upon the nature and extent of the vegetation in the adjacent areas will be of value. Crossroads, bridges, and culverts are responsible in many cases for localized cracks and failures, and information as to their location is important.

The subgrade survey carried on in connection with the condition survey will require inspection of the

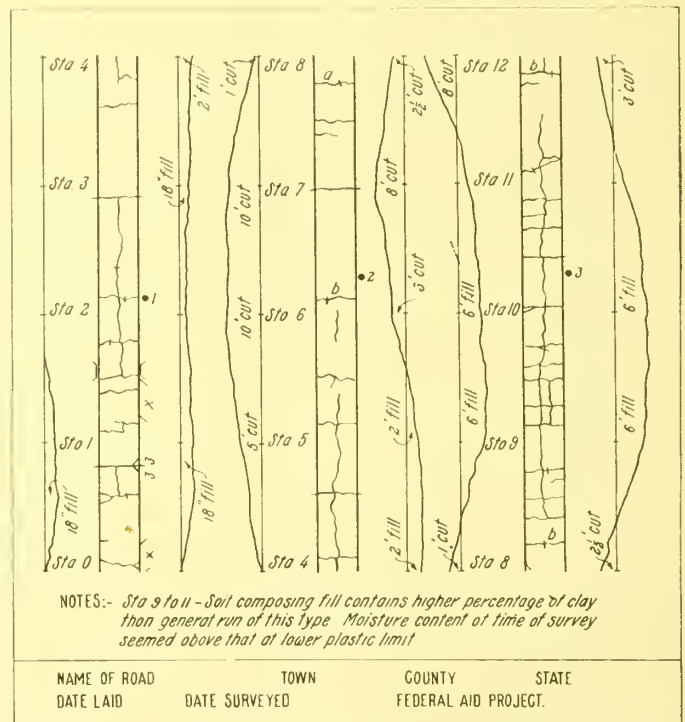


FIG. 2.—SAMPLE OF FIELD RECORD MADE ON ONE OF THE PRELIMINARY PAVEMENT CONDITION SURVEYS

The pavement condition party will consist of a chief, experienced in road construction, and three assistants. Two assistants will measure off and mark the stations on the road, while the engineer walks along the pavement with a log board and specially prepared forms and sketches pavement defects and other features previously mentioned. The fourth member of the party will inspect the subgrade and collect the soil samples. The form which has been prepared contains space for plotting 1,200 feet of pavement survey. (Fig. 2.)

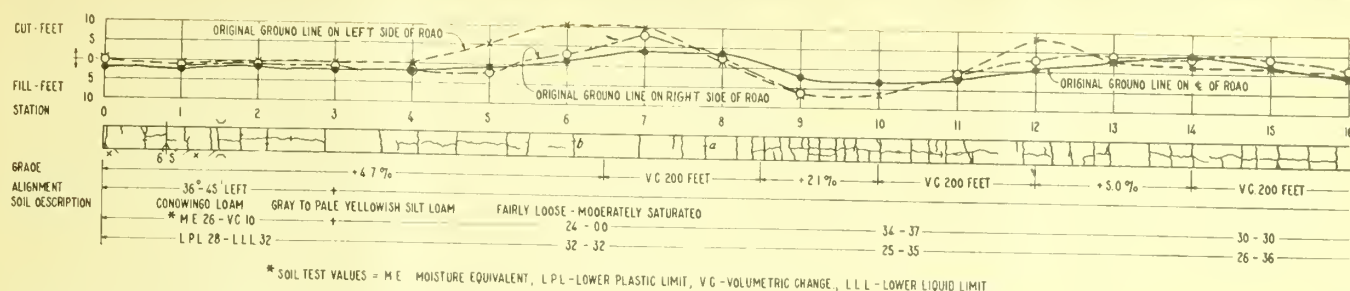


FIG. 3.—SAMPLE OF PERMANENT RECORD SHEET FOR RECORDING RESULTS OF PAVEMENT CONDITION AND SUBGRADE SURVEYS. THE PRESENT ROAD SURFACE IS REPRESENTED BY A HORIZONTAL LINE AND INFORMATION AS TO GROUND SURFACE IS PLOTTED RELATIVE TO THIS LINE

RECORDS TO BE IN FORM SUITABLE FOR PRESENT AND FUTURE USE

Permanent records of surveys are to be made on 24 by 36 inch sheets on which will be shown in detail the condition of the slab, the results of the soil analysis, and other pertinent information secured from the construction plans of the road. This will include the depth and length of the cuts and fills made prior to the construction of the pavement, the beginning and end of relocations, and the grade and alignment changes. Accurate correlation of condition survey information with that on the original road plans will depend on frequent checks between the new and old stationing.

The permanent detailed record will serve a number of purposes. It will be of immediate value as the basis for compiling the summary record and will greatly lessen work on future surveys. Figure 3 shows the details of a portion of such a chart, a single sheet of which is sufficient to record the data for 6,400 feet of pavement.

CONCLUSIONS TO BE BASED ON COMPARISONS OF SUMMARY RECORDS

The summary record is to be the analytical digest of each survey. It will present condensed information from which deductions may be drawn as to the variables in one road as compared with similar variables in

TABLE 1.—Typical summary record of condition survey

Soil type and location	Number of each	Length of pavement	Number of transverse cracks and joints	Average spacing of transverse cracks	Total length of longitudinal cracks	Pavement cracked longitudinally	Number of corner breaks		General breaking		Surface checking; number of checks per linear foot of pavement					Soil tests ¹ (average values)			
							Class 1	Class 2	Class 1	Class 2	None	5	15	30	45	Moisture equivalent	Volume change	Lower plastic limit	Lower liquid limit
Conowingo loam:		Feet		Feet	Feet	Per cent			Sq. ft.	Sq. ft.	Per cent	Per cent	Per cent	Per cent	Per cent	Per cent	Per cent	Per cent	Per cent
Cut.....	8	1,400	39	36	750	52	1				84	10	6			22	11	26	29
Fill.....	8	3,500	108	32	2,520	72	4	3			65	15	14	5	0.6	24	16	26	28
Grade.....	3	1,000	26	39	510	51	3				57	17	14	12		25	17	23	27
Total.....		5,900	173	34	3,780	64	8	3			69	14	12	5	0.3	24	15	26	28
Manor loam:		Feet		Feet	Feet	Per cent													
Cut.....	14	4,275	86	50	1,370	32	2				35	39	27	7	2	20	5	28	29
Fill.....	12	4,675	70	67	3,000	64		1			40	26	22	10	2	21	5	29	29
Grade.....	4	1,900	50	38	950	48					67	24	6	3		19	10	23	23
Total.....		10,850	206	53	5,320	49	2	1			49	31	21	7	2	21	6	25	26
Chester loam:		Feet		Feet	Feet	Per cent													
Cut.....	9	2,000	32	62	1,100	55	2	1			30	47	11	6	6	22	12	26	27
Fill.....	8	3,625	58	63	2,430	67					53	20	17	8	2	21	9	27	27
Grade.....	4	1,125	17	66	780	69					27	15	23	29	6	29	23	27	27
Total.....		6,750	107	63	4,440	66	2	1			42	27	16	11	4	22	12	27	27
Louisa loam:		Feet		Feet	Feet	Per cent													
Cut.....	2	500	6	83	330	67					70	30				19	7	27	27
Fill.....	2	1,000	14	71	680	68					83	17				25	10	32	32
Grade.....		200	4	57	200	100					18	82							
Total.....		1,700	24	72	1,210	71	0				71	29				23	9	30	30
Iredell loam:		Feet		Feet	Feet	Per cent													
Cut.....	2	1,150	23	50	720	65					32	50	11	7		28	27	27	34
Fill.....	4	2,150	40	54	2,020	94	1	3			34	40	17	9		23	18	22	26
Grade.....																			
Total.....		3,300	63	53	2,740	83	1	3			33	44	15	8		25	20	24	29
Granville loam:		Feet		Feet	Feet	Per cent													
Cut.....	1	350	10	37	310	91	1				34	31	31	4		18	10	17	22
Fill.....	1	450	4	112	450	100					55			4.5		22	12	22	26
Grade.....	1	200	3	66	200	100					80	15	5			21	10	23	28
Total.....		1,000	17	60	960	96	1				61	14	12	3.5		20	11	21	25
All types:		Feet		Feet	Feet	Per cent													
Cut.....	36	9,675	196	49	4,580	48	6	1			37.8	36	18	5.2	2.0				
Fill.....	35	15,400	294	52	11,100	72	5	7			52.0	23	16	7.5	1.3				
Grade.....	13	4,425	100	44	2,640	60	3				52.5	23	12	11	1.5				
Total.....		29,500	590	50	18,320	62	14	8			47.5	27.5	16	7.3	1.6				

¹ Routine soil test procedure described on page 2 was adopted after this survey was made.

another. The form and extent of the summary record is shown in Table 1. The information will be obtained directly from the permanent record charts by scaling and totaling the lengths of the cuts, fills, and grade sections in each soil type. The original ground line will be used in the computation of vertical distances. In addition to the information shown in Table 1, there will be included data as to the geological character of the soil.

On many surveys, information concerning the effect of the soil type and of cuts and fills upon slab or surface condition will include a summary record of—

1. The width of cracks and joints.
2. The raveling at cracks and joints.
3. The faulting of slabs.
4. The sealing of the surface.

On other surveys information on the slab condition for different percentages of varying grades and alignment may be necessary for the proper interpretation of the results. The effect of depth of cut or fill may also be shown in the summary record.

The data to be contained in the summary records will permit relationships to be established between the slab or surface condition and the results of the soil tests. These relationships may be shown conveniently in diagrammatic form. For example, slab condition, percentage of longitudinal cracking, or number of corner breaks may be compared with the moisture equivalent or other test values.

SURVEYS TO BEGIN AT AN EARLY DATE

A field survey has been made in Virginia to determine the practicability of the methods outlined and it was found that the procedure is such that reasonably rapid work can be done in the field and laboratory. At the present time preliminary work is being done in cooperation with the Maryland State Roads Commission on the selection of roads for detailed survey during the coming summer. It is anticipated that surveys will be made in other States also.

All of the surveys undertaken will be carried on in cooperation with the State highway departments. The procedure as herein set forth is more or less tentative pending suggestions from the committee on tests of the American Association of State Highway Officials, the committee on structural design of roads of the Highway Research Board, and from all engineers interested in the highway problem. It is only by criticism and suggestion at this time that the final program of the investigation can be so framed that the results obtained will have the utmost value.

In addition to criticisms and suggestions, information on the following will be very much appreciated:

1. Location of roads where special drainage, subgrade treatment or subbase has been used under part of the surface.
2. Location of roads where widely different types of soils are found.
3. Existence of maintenance costs for roads which are similar in all respects except subgrade.
4. Existence of pavement condition survey data without regard to scope or purpose for which they were obtained.

SEVERAL STATES NOW CONSIDER SUBGRADE INFLUENCE

An examination of current practice in regard to subgrades seems pertinent at this point. Many States give little or no consideration to the subgrade, using

the same pavement designs for all conditions of support. Other States consider subgrade influence and compensate for it in various ways. The following information has been collected as to practice in some of the States:

Arizona.—Design of pavements dependent on subgrade rainfall, and temperatures. Type and thickness of pavement varied. Silty sand subgrade stabilized by adding mixture of gravel, caliche, and clay. Very fine dust subgrade (high cementing value) surfaced with clean, coarse gravel. Adobe soils surfaced with volcanic cinders, gravel, or similar available material. In some cases this treatment is used where surfacing is contemplated in the future.

California.—Subgrade soil tests are part of routine work on new construction jobs. Soils of low shrinkage and low moisture equivalent value are given preference for embankment. When moisture equivalent exceeds 30 per cent and the lineal shrinkage exceeds 5 per cent, subgrades are stabilized by the addition of sand, decomposed granite, or similar material.

Oregon.—Subgrade conditions generally determined by inspections by division engineers but in some cases laboratory tests are made. In some cases of questionable subgrade, sand sub-base (thickness dependent on soil shrinkage values) is used while in others (soft places or adobe) slab design is changed by adding reinforcement and reducing joint spacing.

Washington.—Subgrades are inspected by the construction engineer from the headquarters office and the district engineer. Questionable subgrades are compensated by increasing the thickness of paving, increasing or adding reinforcement, reducing the slab length or by adding gravel sub-base. Soft spots are dug out and replaced with gravel. Tile drains are used in wet and spongy places.

Colorado.—Subgrades are sampled and tested before pavement construction. Subgrades having a lineal shrinkage in excess of 5 per cent receive sand treatments of from 2 to 10 inches, depending on the extent to which the moisture equivalent exceeds 20 per cent.

Minnesota.—Clay subgrades are stabilized by the addition of sand and gravel and sand subgrades by an admixture of clay. In swampy sections fills are made with coarse-grained upland material, while deep wide ditches and a generous amount of tile are used to remove free water. Frost-boil failures are treated with admixtures of gravel. Special reinforcement is added to concrete pavements when the soil seems especially unstable. In some cases subgrades are oiled.

Kansas.—In isolated cases a sand sub-base has been used on gumbo soil.

Massachusetts.—Subgrade conditions are determined by visual inspection during the spring break-up, examination of material from test pits along the project, and by inspection during construction. Concrete pavements are not varied but foundations of stone and gravel and drainage are. Side drains are effectively used.

Rhode Island.—Subgrade conditions are determined by inspections made in the spring when frost is leaving the ground. When the road is on a new location samples of subgrade are secured for test. Sub-bases of gravel are used under penetration macadam or concrete roads as compensation for questionable subgrades.

Connecticut.—Double reinforcement in the slab and either sand, gravel, or stone sub-base connected to underground drains is used in locations where heaving is expected.

Maine, New Hampshire, New Jersey, and New York also compensate for questionable subgrade. This compensation includes gravel and stone sub-base, bituminous treatment of the subgrade, and steel reinforcement of the slab.

LARGE SAVINGS MAY RESULT FROM GREATER KNOWLEDGE OF SUBGRADES

Before undertaking an investigation of this kind it is pertinent to inquire as to the value of the results. It is not possible, of course, to form even an approximate estimate of the value of results in general, but the following instances will indicate the substantial value of the information it is proposed to obtain:

On one of the older Maryland roads (Beacon Hill-Elkton) about 30 per cent of one section is badly broken while an adjoining section is intact, variation in subgrade being responsible for the difference.

(Continued on p. 20)

THE TREND OF HIGHWAY DESIGN

Reported by A. G. BRUCE, Highway Engineer, and R. D. BROWN, Associate Highway Engineer, Bureau of Public Roads

TO ONE familiar with the practice of the State highway departments the changes which have been made in the design of highways during recent years appear as no less remarkable than the notable changes in the volume and character of traffic by which they have been called forth. The general character of these changes is perhaps already well known to all highway engineers, but it is thought that a review in some detail of the practices now employed by each of the State highway departments, and their evolution during the past several years will be of interest as indicating the various shades of opinion on doubtful points and the degree of standardization obtained.

Opportunity for such a review is presented by the plans received from all States by the Bureau of Public Roads for Federal-aid projects, and the following analysis is based upon a large number of such plans received during the last six years:

Crown.—Nearly all the States now require that hard-surfaced pavements shall meet a surface trueness test, and this has resulted in greater care in finishing and has permitted a reduction in crown until the pavement now appears to be almost flat. The crown on cement concrete, brick, sheet asphalt, or any of the bituminous concrete pavements on rigid foundations is rarely more than one-fourth inch per foot. Some States have adopted one-eighth inch per foot, and two States use one-tenth inch per foot. The trend of practice is toward the latter amount, and it is probable that a 1-inch crown for a 20-foot hard pavement will soon be the prevailing practice. On sand-clay, gravel, and waterbound macadam roads the prevailing crown is one-half inch per foot and on bituminous macadam roads three-eighth inch per foot is quite general.

Curvature.—The trend toward easy horizontal curves and longer vertical curves is very apparent. A few years ago curves of 150 and 200 foot radius were common because of right-of-way difficulties and grading costs. Both engineering opinion and public sentiment have changed considerably and now the radius is seldom less than 500 feet, and some States have adopted 1,000 feet as the standard radius for main highways. Tangents at least 100 feet long are usually provided at bridges and between reverse curves.

Vertical curves.—The length of vertical curve has increased with the volume and speed of traffic which have made necessary greater sight distances. A rule used in many cases is that the length of vertical curve in stations shall equal one-half the algebraic difference of the approach grades in per cent. This rule results in a sight distance of approximately 450 feet, but on roads of primary importance the minimum sight distance on convex vertical curves is generally 500 feet and on secondary roads not less than 350 feet. Vertical curves are now used at all changes of grade on hard pavements, and on gravel and macadam, where the algebraic difference in grades is more than 0.5 per cent.

Grades and alignment.—The 1926 designs showed a continued upward trend of standards of alignment. The alignment is becoming straighter. Relocation of

crooked roads and the use of long, easy curves are becoming common. The grades have not been appreciably reduced; in fact, there appear to be more short, steep grades than heretofore. This is due in part to the fact that engineers now prefer to avoid sharp curvature even at the expense of steeper grades. There is no uniformity in the ruling grades adopted by the various States, but 5 per cent seems to be the prevailing practice. Grades of 7 per cent, however, are not uncommon, and in the Appalachian Mountain section short 9 per cent grades are still considered necessary.



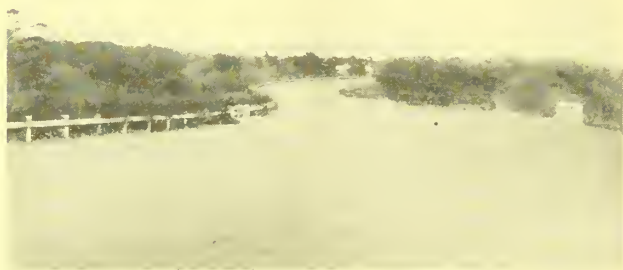
A MODERN CONCRETE PAVEMENT. THE LONGITUDINAL JOINT PREVENTS IRREGULAR LONGITUDINAL CRACKS AND ACTS AS A GUIDE LINE FOR THE TRAFFIC

Compensation of grades.—There is no general acceptance of the theory that grades should be compensated for curvature on the modern highway designed for motor-driven traffic. This feature in grade design is, however, common in the Pacific Coast States.

Good practice seems to indicate that grades in excess of 5 per cent should be compensated on curves of less than 500-foot radius. The amount of compensation necessary depends upon several factors, including the length of the approach grade and its rate, the degree of curvature, length of curve, the amount of superelevation, the sight distance, and the general character of traffic expected. A simple rule that appears to have

considerable merit is to compensate grades over 5 per cent on all curves of less than 500-foot radius at the rate of 0.5 per cent for each 50-foot reduction in the radius below 500 feet.

Superelevation and widening.—Superelevation and widening of curves is now universally practiced, but there is no agreement in formula or method in the various States. A few States superelevate the roadway on all curves sharper than 1° , but the prevailing practice is decidedly more conservative. The average practice seems to be to superelevate all curves of less than 2,000-foot radius and to widen all curves of less than 800-foot radius. A maximum superelevation of 1 inch per foot of width, and a maximum widening of 4 feet represent the prevailing practice. The sub-



SUPERELEVATION AND WIDENING ON A CURVE IN A GRAVEL ROAD

committee on design of the American Association of State Highway Officials recommends, as a rule for widening, the formula suggested by J. T. Voshell, district engineer of the Bureau of Public Roads:

$$W = 2(R - \sqrt{R^2 - L^2}) + \frac{35}{R}$$

In which W = Widening in feet.
 R = Radius of curve in feet.
 L = Wheel base of vehicle in feet
 (20 feet recommended).

The same committee recommends for superelevation the formula $E = 0.067 \frac{V^2}{R}$

In which E = Superelevation in feet per foot of width.
 V = Velocity in miles per hour (35 recommended).
 R = Radius of curve in feet.

Width.—Eighteen feet still predominates as the width of pavement on Federal-aid projects although pavements 20 feet wide and wider are not uncommon. Of the 3,643 miles of surfaced Federal-aid road brought to completion in the six months period preceding February 1, 1927, approximately half had 18-foot surfacing. Eight hundred and thirty-one miles were less than 18 feet wide, 1,809 miles were of 18-foot width, 900 miles were between 19 and 24 feet wide, 79 miles between 25 and 30 feet wide, and there were 24 miles of unusual construction not falling in the above classification.

Subgrade.—A majority of the States give little or no consideration to variations in the character of the subgrade, using the same pavement design throughout

each project. A few consider subgrade influence and compensate for it in varying degrees and manners.

The method of procedure in these States and the general subject of subgrade investigations is discussed in another report by the bureau.¹

Guard rail.—In the minor features of highway design, such as culverts, guard rail, curbs, shoulders, etc., there do not appear to be any marked developments during the past year. The trend in guard-rail design continues to be toward the steel cable and woven wire and away from the wooden type. Where local conditions justify wooden guard rail, the plan of placing the rail at the elevation of the vehicle hub is gaining in favor. Some States are using steel and concrete guard-rail posts, but the prevailing practice is to use wooden posts. Some have adopted a new arrangement for fastening the wire cable to the post. Instead of running the cable through the post or bolting it in direct contact, a casting is used which holds the cable about 4 inches in front of the post. The claim is made that this arrangement adds a measure of safety, as vehicles will be less likely to hit the post.

Curbs.—Curbs at the edges of the roadway on bridges are being increased in height as a safety measure and as a protection to the bridge structure. Curbs 9 to 12 inches in height are used. Some States use even higher curbs and batter the face so that the vehicle hub will not scrape. Several bridge failures have occurred as a result of vehicles colliding with a main bridge member, and signs of collisions are so common on bridges as to indicate the necessity of protection.



DEVICE FOR OFFSETTING CABLE WIRES FROM SUPPORTING POSTS

Shoulders.—There is a decided trend toward wider shoulders on rural highways. On any main route the entire surface is needed to accommodate moving traffic. Even where traffic is light a vehicle parked on any portion of a surface designed to accommodate two lanes of traffic constitutes an element of danger, and if such parking is on a location with restricted sight distance the danger is a most serious one. The stopping of vehicles on account of breakdowns and other reasons is unavoidable and wider shoulders to accommodate such cases are justified. The greater width of graded roadway will permit surface widening on a consolidated foundation when required. At least 80 per cent of the current plans provide for shoulders more than 4 feet in width. Most of them provide for 5 feet, and a few require 6 feet or more.

Grade crossing elimination.—The railroad grade crossings are recognized as one of the most important factors in the highway problem, and the public is thoroughly aroused to the necessity for reducing this

¹ See Subgrade Studies of the Bureau of Public Roads on page 1 of this issue.

danger. It is recognized that all crossings can not be eliminated immediately and that an annual program must be worked out, abolishing the crossings in the order of their relative danger and taking into consideration the volume of rail and highway traffic.

During the past year 414 grade crossings have been eliminated on Federal-aid projects, making a total of 1,794 since Federal-aid work started. This is about 34 per cent of the grade crossings encountered. Of the crossings eliminated 1,076 were by relocation and the balance by means of overpass or underpass. Of the crossings eliminated during 1926, 202 were by relocation of the highway.

Road types.—Figures 1 and 2 show graphically the trend of type selection on Federal-aid projects. The data are shown by Federal fiscal years ending June 30 of the designated year. The figures would not be materially different had calendar years been used instead of fiscal years. Figure 1 shows the total mileage of the various types of completed Federal-aid projects, and also the mileage completed during the fiscal year 1926. Gravel easily predominates with 38.7 per cent of the total, and cement concrete is second with 20.3 per cent. These two types also predominated in 1926 with almost the same ratios, namely, gravel 38 per cent and cement concrete 24 per cent. Sand-clay mileage is 9.3 per cent of the total but only 6.3 per cent of the 1926 mileage, indicating that this type is not holding its former popularity. The mileage of this type has been steadily decreasing on Federal-aid work since 1922, and for new work is now practically confined to Alabama, Georgia, and South Carolina. Waterbound macadam also shows a decreasing popularity, being 2 per cent of the grand total and only 0.7 per cent of the 1926 mileage; but bituminous macadam forms a larger percentage of the 1926 work than of the total mileage. Bituminous concrete maintains approximately the same position in the 1926 work that

it occupies in the total mileage, but brick dropped considerably. Comparison of the mileage completed each year since 1920 is shown in Figure 2.

Concrete pavement design.—Since the use of Portland cement concrete for pavement surfaces is a comparatively recent development, it is natural that more changes are to be found in the design of this type of pavement than in other types. Tables 1 and 2 summarize the general practice in 1926. The most important change is in the shape of the cross section. Prior to the season of 1921 all Federal-aid concrete projects provided for pavements of uniform thickness or of less depth at the edges than at the center. In 1921 the State Highway Department of Arizona submitted three projects involving the unique idea of constructing concrete pavements with the edges thicker than the center. During 1921 and 1922 the Pittsburg and Bates test roads were constructed and the results of the tests were widely published, with the result that in 1922 over 5 per cent of the concrete pavement designs for Federal-aid projects were of the thickened-edge type. In 1923 the percentage of thickened-edge designs increased to 46 per cent, in 1924 to 73 per cent, in 1925 to 80 per cent, and in 1926 to 81 per cent. In mileage of roads the percentage of the thickened-edge type would be much greater because the uniform-thickness and thin-edge designs are mostly on short projects.

While the thickened-edge section has been almost unanimously accepted in principle by the State highway departments, there is still a notable lack of uniformity in the dimensions of the section used in various States which can not be wholly accounted for by varying requirements of loading and subgrade conditions. The difficulty encountered in properly shaping and consolidating the subgrade has had considerable effect in producing these variations. A study of the designs submitted to the bureau during 1926 shows that in

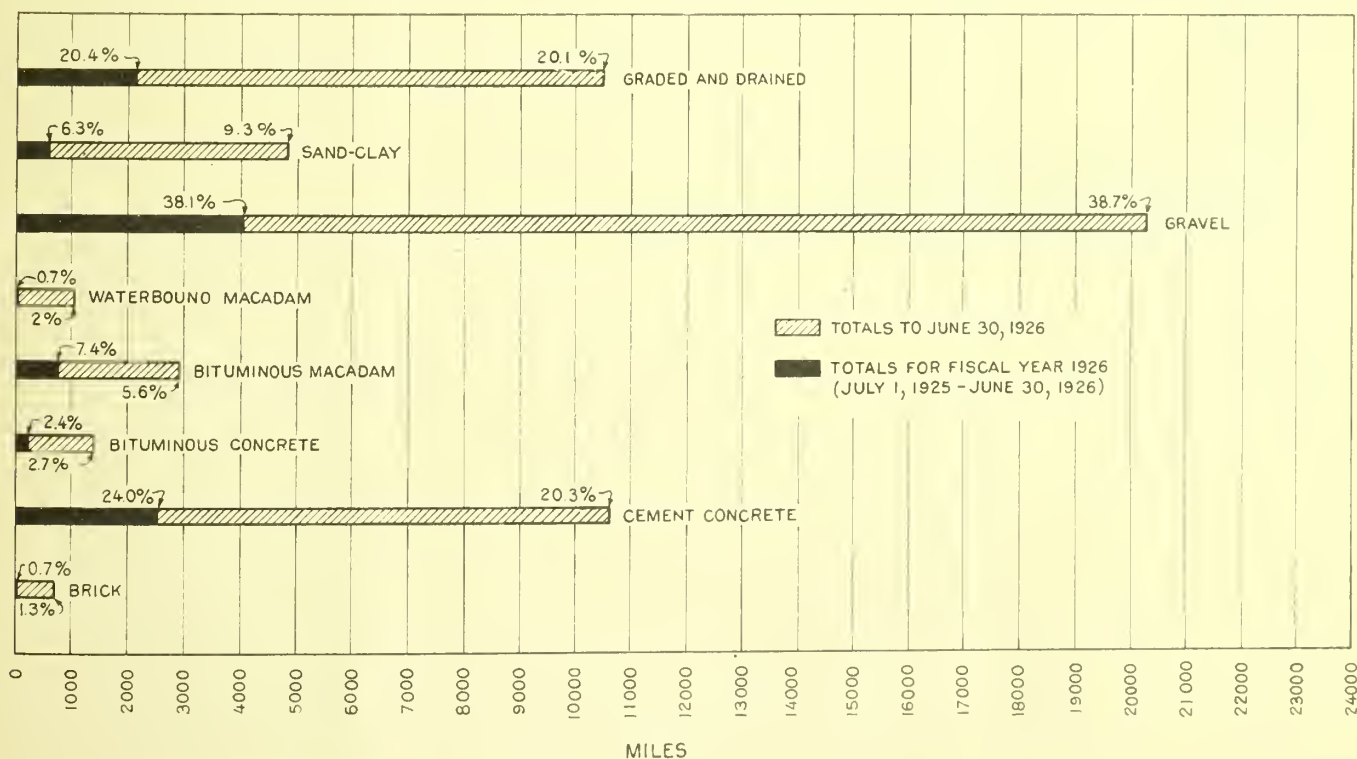


FIG. 1.—MILEAGE OF FEDERAL-AID ROADS COMPLETED BY TYPES TO JUNE 30, 1926, AND DURING THE FISCAL YEAR 1926

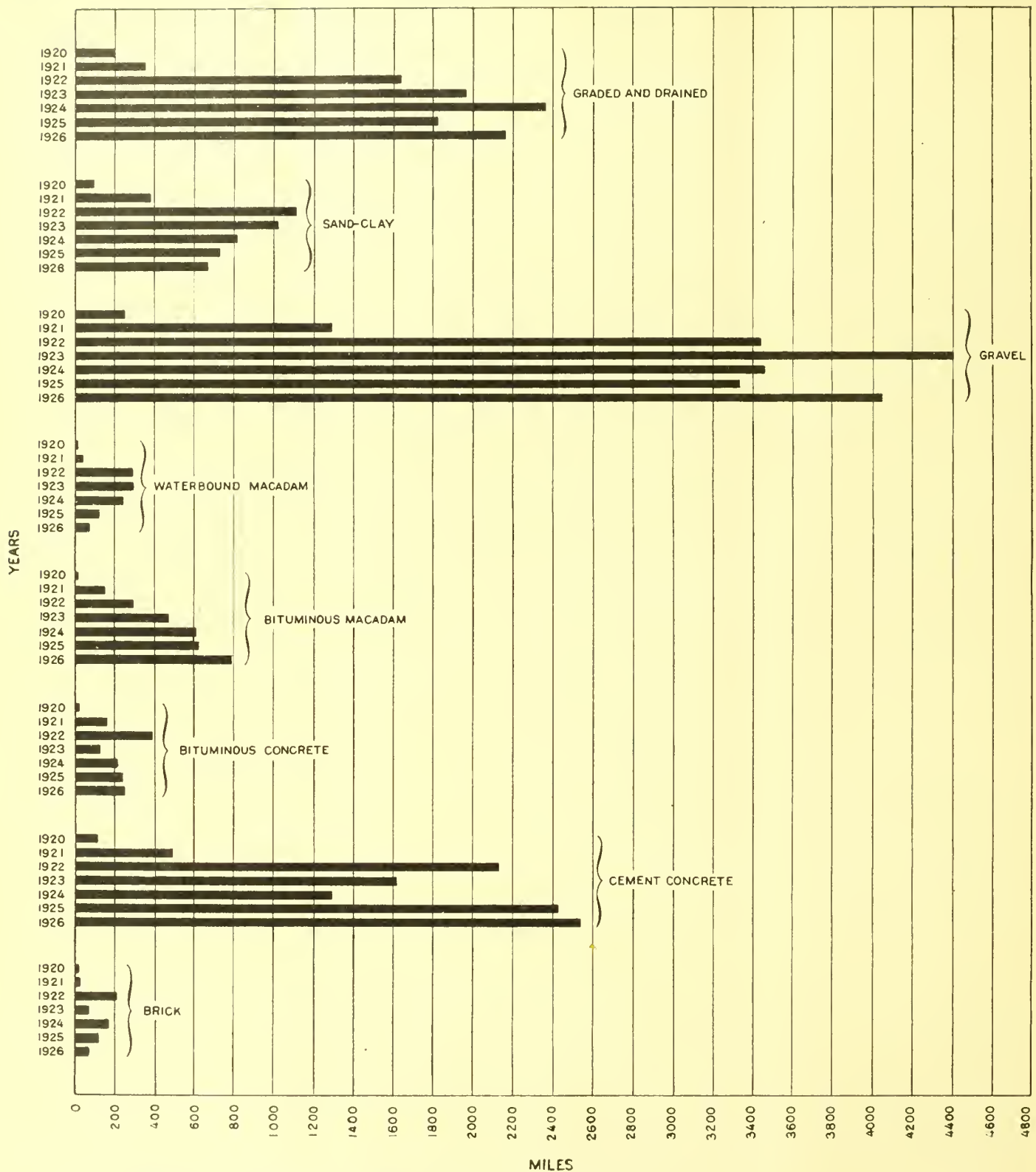


FIG. 2.—MILEAGE OF EACH TYPE OF ROAD CONSTRUCTED WITH FEDERAL AID EACH FISCAL YEAR SINCE 1920

15 States the increase in thickness is made in the outer 2 feet of the pavement, while in 10 States the increase is made in the outer 3 feet. In 4 States the transition is made in the outer 4 feet. Six States obtain the thickened edge by using a curved subgrade and a surface crown of longer radius, and one State uses a two-plane subgrade so that the increase in depth is at a uniform rate from the center to the edges.

Next in importance to the general adoption of the thickened-edge design is the adoption of the longitudinal center joint in concrete pavements by a large majority of the State highway departments. Thirty-four States now definitely require a longitudinal joint and it is expected that the coming season will show an increase in this practice. The longitudinal joint, in addition to effectually preventing irregular and un-

sightly longitudinal cracking, acts as a permanent traffic line and permits of half-width construction. A number of States now require that the pavement be constructed one-half at a time in order to avoid detouring traffic during the construction and curing period. In most cases the joint is formed by a deformed plate of 16 or 18-gauge metal which produces a tongue-and-groove connection. A majority of the States also require dowels across the longitudinal joint but there is no general agreement as to whether or not these dowels should be bonded. In some States deformed bars are used to insure a bond and in others precautions are taken to prevent bonding.

The question as to whether or not expansion joints are required in concrete pavements is being given considerable study, as is the question of their proper spacing. The majority of States are using expansion joints at more or less regular intervals but a considerable

joint. There does, however, seem to be a general agreement that when dowels are used, provision should be made for free movement of the concrete by preventing the bonding of both ends of the dowels.

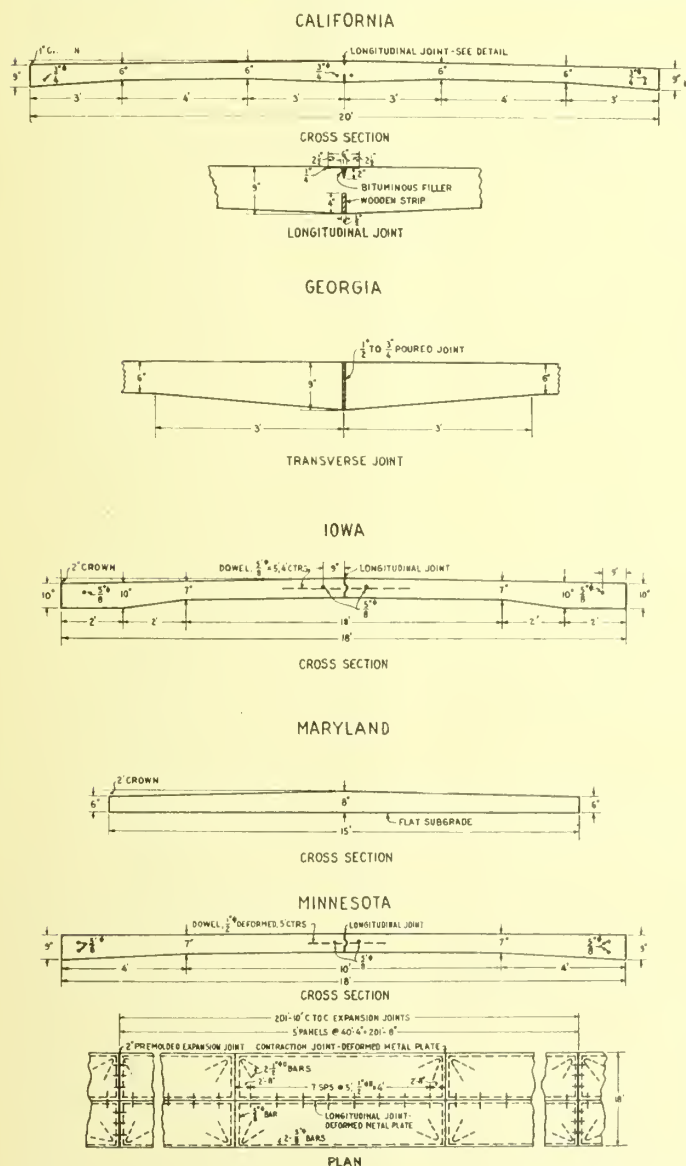


FIG. 3.—SPECIAL FEATURES OF CONCRETE PAVEMENT DESIGN USED IN 1926

number of States do not consider them necessary. As yet no general agreement has been reached among the States using expansion joints as to their proper spacing and width, or the necessity of dowels across the

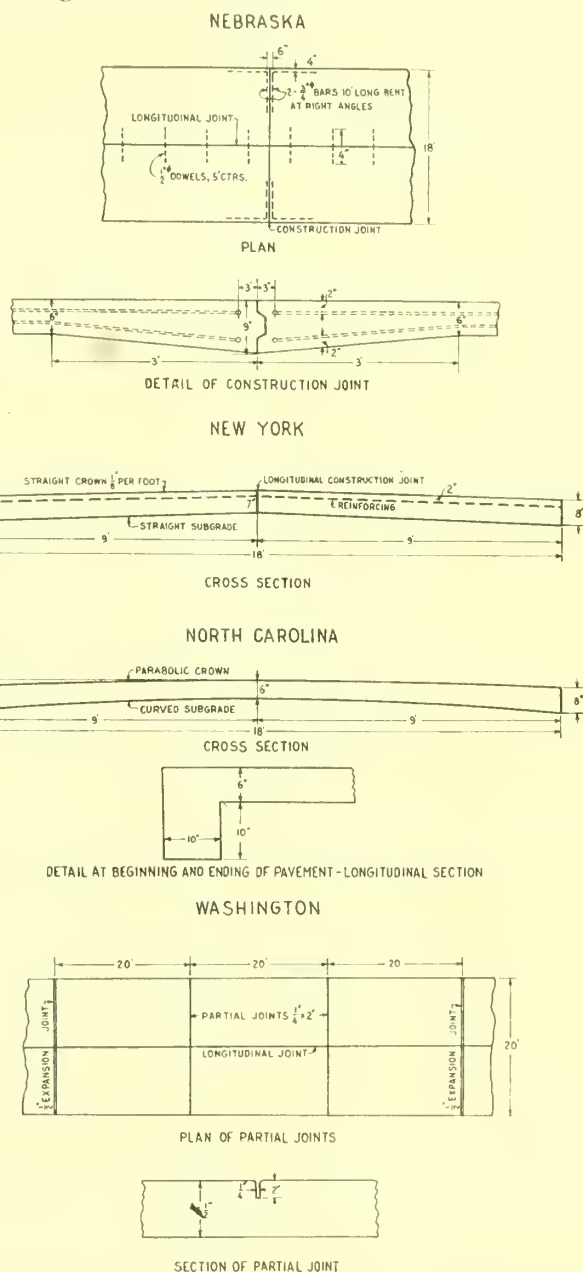


FIG. 4.—SPECIAL FEATURES OF CONCRETE PAVEMENT DESIGN USED IN 1926

Little change is noted in the proportions of cement to total aggregate in the pavement mixtures but there is a growing tendency to vary the proportions of fine to coarse aggregates to produce dense concrete. Considerable emphasis is being placed on the amount of water used in the mix and relatively dry mixes are now the rule rather than the exception. Admixtures do not appear to be gaining in popularity since only two States use an admixture as a standard requirement. The use of special cement or special methods of manipulation to obtain high early strength is coming into use for emergency work, particularly in the construction of bridge floors, but does not appear to be considered for ordinary concrete pavement construction because of

TABLE 1.—General features of design of cross section of concrete pavements on Federal-aid projects submitted in 1933

State ¹	Pavement cross section				Crown	Mix proportions	Reinforced type			Plain type	
	Width	Thickness		Thickened-edge width			Bars	Mesh	Location		Edge bars
		Edge	Center								
		Inches	Inches								
Alabama	18	9	6	9	3	2-inch, curved	1:2:3	Lbs. per 100 sq. ft.		No steel	
Arizona	18	9	6	9	2	1½-inch, curved	1:2:3½			Dowels only	
Arkansas	18	9	6	9	2	2½-inch, curved	1:2:3½			Two ¾-inch diameter smooth edge bars	
California	20	9	6-9-6	9	3	1-inch, curved	1:2:4 (approximate)			Four ¾-inch diameter deformed edge bars	
Colorado	18	9	6½	9	3	1½-inch, parabolic	1:2:3			Dowels only	
Connecticut	18 to 20	8	8	8	3	2½-inch, circular	1:1¾:3½ or 1:2:3½	56 to 63	Top or bottom	Do	
Delaware	18	8	6	8	Circular	2-inch, circular	1:2:4		2 inches from top	Do	
Florida	18	9	6	9	3	2½-inch, curved	1:2:4 or 1:2:3½	40	do	No steel	
Georgia	18	9	6	9	3	1½-inch, curved	1:2:3½	44		Dowels only	
Idaho	18	8	6	8	2	1-inch, curved	1:2:3			Two ¾-inch diameter smooth edge bars, painted or oiled	
Illinois	18	9	6	9	3	2-inch, circular	1:2:3½			Do	
Indiana	18	9	7	9	2½	2-inch, curved	1:2:3			Four ½-inch diameter edge bars	
Iowa	18	10	7	10	2	2-inch, curved	1:1½:3 to 1:2:3½			Four ¾-inch diameter smooth edge bars	
Kansas	18	9	6	9	4	2-inch, parabolic	1:2:3½			Two ¾-inch diameter smooth edge bars, painted or oiled	
Kentucky	16	9	6	9	2	2-inch, curved	1:2:3½	42 (alternate types)	2 inches from top		
Louisiana	20	9	6	9	2	2½-inch, curved	1:2:3½	58	do		
Maine	20	9	7	9	4	1½-inch, curved	1:2:3½	91	5 inches from top	No steel	
Maryland	15	6	8	6		2-inch, circular	1:2:4 (approximate)	22 to 25	2 inches from top		
Massachusetts	20	8	8	8		2½-inch, circular	1:2:4	95	2 inches from bottom		
Michigan	20	9	7	9	3	2-inch, curved	1:2:3½	90 (alternate types)	3½ inches from top		
Minnesota	18	9	7	9	4	1-inch, curved	1:2:3½			Dowels only	
Mississippi	18	9	6	9	2	2-inch, circular	1:2:3			Six ¾-inch diameter edge bars	
Missouri	18	9	6	9	2	1½-inch, curved	1:1½:3 to 1:2:3½			Two ¾-inch diameter smooth edge bars	
Nebraska	18	9	6	9	3	1½-inch, curved	1:1½:3			Two ¾-inch diameter smooth edge bars, painted or oiled	
New Hampshire	18	9	6	9	2	2-inch, curved	1:1½:3 (approximate)	66	2 inches from top	Corner bars, ¾-inch diameter	
New Jersey	20	8	8	8	2	2-inch, parabolic	1:1¾:3½ to 1:1½:3	28 to 80 (alternate types)	Top and bottom		
New Mexico	18	9	6	9	2	1½-inch, parabolic	1:2:3 (approximate)	25	2 inches from top	No steel	
New York	18	7	6	7	9	1½-inch, straight	1:1½:3	80 (alternate types)	do	4½-inch diameter (diameter)	
North Carolina	16 to 18	{ 4 6 5 8	6	4 6 5 8	None. Parabolic.	2-inch, parabolic	1:1½:3 to 1:2:4	72 to 74	2½ inches from top	Dowels only	
North Dakota	20	9	6	9	3	3½-inch, circular	1:2:3½	47	2½ inches from bottom	Two ¾-inch diameter smooth edge bars, painted or oiled	
Ohio	18	9	7	9	2	1½-inch, curved	1:2:3	As shown	2 inches from top	Dowels only	
Oklahoma	18	9	6	9	2	2-inch, curved	1:2:3½	32	4 inches from top	Do	
Oregon	20	9	7	9	3	1½-inch, curved	1:2:3 (approximate)		2 inches from top	No steel	
Pennsylvania	20	8	6	8	Curved	2-inch, circular	1:2:3 (approximate)		do	Dowels only	
Rhode Island	18	8	8	8	Curved	2-inch, curved	1:2:3 (approximate)	32 to 45	2 inches from top		
South Carolina	18	8	6½	8	Parabolic	2-inch, parabolic	1:2:4	42	do	Do	
Tennessee	20	8	6	8	Curved	1½-inch, parabolic	1:2:3½	25	As shown	No steel	
Texas	18	9	6	9	2	1½ to 2¼ inch, curved	1:2:3 (approximate)	44	As shown on plans	Dowels only	
Utah	18	9	6	9	2	1½-inch, curved	1:2:3 (approximate)		2 inches from top	Two ¾-inch square edge bars	
Vermont	18	7	7	7	2	2-inch, curved	1:2:4	40½	Top and bottom	Do	
Virginia	18	8	6	8	Curved	2½-inch, curved	1:2:4 (approximate)			No steel	
Washington	20	9	6½	9	2	1½-inch, curved	1:2:3			Dowels only	
West Virginia	18	7	7	7	2	2½-inch, parabolic	1:1¾:3½ to 1:2:3½			No steel	
Wisconsin	20	9	6½	9	4	1-inch, curved	1:2:4 (approximate)			Dowels only	

¹ Four States, Montana, Nevada, South Dakota, and Wyoming submitted insufficient mileage to justify inclusion in the table.² Mixture of 0.1 cubic foot of hydrated lime per bag of cement.³ Mixture of 0.08 cubic foot of hydrated lime per bag of cement.⁴ Unreinforced section⁵ Reinforced section.

TABLE 2.—General features of design of joints in concrete pavements on Federal-aid projects submitted in 1926

State 1	Longitudinal joint			Transverse joint			Dowels		Special features
	Type	Gauge or width	Type	Spacing	Width	Filler	Longitudinal joints	Transverse joints	
Alabama	Not required		Expansion	Feet	Inches	Prepared bituminous.	1½-inch diameter, 5 feet c. to c.	None	
Arizona	As shown on plans		do	40	¾	do	4 feet by ½-inch diameter, deformed, 5 feet c. to c.	do	
Arkansas	do		do	50	½	Prepared or poured bituminous.	None	Two, 4 feet by ½-inch diameter	
California	Submerged. (See fig. 3.)		do	50	½ to ¾	do	4 feet by ½-inch diameter, smooth, 5 feet c. to c.	Ten, 2 feet by ¾-inch diameter, smooth, one end free.	See Figure 3 for longitudinal joint and cross section.
Colorado	Deformed metal plate	18	do	60	½	Prepared bituminous.	4 feet by ½-inch diameter, smooth, 5 feet c. to c.	do	
Connecticut	Prepared or poured bituminous	¼-in.	do	40	½	Prepared or poured bituminous.	4 feet by ½-inch diameter, deformed, 5 feet c. to c.	Seven, 4 feet by ½-inch diameter, smooth, one end free.	Reinforcing in top on cuts, in bottom on fills.
Delaware	Deformed metal plate	16	Construction	(1)			4 feet by ½-inch diameter, deformed, 5 feet c. to c.	Size and spacing not shown, one end free.	Admixture of hydrated lime.
Florida	Not required		Expansion	(1)		Prepared or poured bituminous.	2 feet by ½-inch diameter, 21 inches c. to c.	None	See Figure 3 for thickened transverse joint.
Georgia	do		do	(1)		Poured bituminous	4 feet by ½-inch diameter, deformed, 5 feet c. to c.	do	
Idaho	Deformed metal plate	14	do	30	¾ to ½	Prepared bituminous.	4 feet by ½-inch diameter, deformed, 5 feet c. to c.	do	
Illinois	do	16	Construction	(1)			4 feet by ¾-inch diameter, deformed, 5 feet c. to c.	do	
Indiana	do	16	do	(1)			4 feet by ¾-inch diameter, deformed, 5 feet c. to c.	Six, 4 feet by ¾-inch diameter, smooth, one end free.	
Iowa	do	18	do	(1)			5 feet by ¾-inch diameter, 4 feet c. to c.	Nine, 2 feet by ¾-inch diameter, smooth, one end free.	
Kansas	do	18	Expansion	150	¾	Prepared bituminous	4 feet by ½-inch diameter, deformed, 5 feet c. to c.	None	See Figure 3.
Kentucky	do	16	do	30	½	do	do	do	Tongue and groove construction joint.
Louisiana	do	16	do	30	½	Prepared or poured bituminous.	2 feet by ½-inch diameter, smooth, 3½ feet c. to c., one end free.	Seven, 4 feet by ½-inch diameter, smooth, one end free.	Half-width construction required.
Maine	Poured bituminous filler	(2)	do	40	½	Prepared bituminous.	3½ feet by ½-inch square, deformed, 5 feet c. to c.	Eight, 2 feet by ½-inch diameter, smooth, one end free.	Do.
Maryland	Not required		Construction	(1)			3½ feet by ½-inch square, deformed, 5 feet c. to c.	do	
Massachusetts	Poured bituminous filler	(2)	Expansion	75 to 100	1	Poured bituminous	5 feet by ½-inch diameter, 5 feet c. to c.	Eight, 4 feet by ½-inch square, smooth, one end free.	See Figure 3.
Michigan	Deformed metal plate	16	do	20½	2	do	5 feet by ½-inch diameter, deformed, 5 feet c. to c.	do	Half-width construction and admixture.
Minnesota	do	16	do	50	¼ to ½	Prepared or poured bituminous.	do	do	See Figure 3.
Mississippi	do	18	do	(1)			4 feet by ½-inch diameter, 5 feet c. to c.	do	
Missouri	do	16	Construction	(1)			do	do	See Figure 4 for sketch of thickened, tongue and groove joint.
Nebraska	Deformed metal plate	18	do	(1)			4 feet by ½-inch diameter, 5 feet c. to c.	do	Half - width construction required.
New Hampshire	Poured bituminous filler	(2)	Expansion	50	¼ to ¾	Prepared or poured bituminous.	None	Eight, 2 feet by ½-inch diameter, smooth, one end free.	Double line of reinforcing.
New Jersey	Prepared bituminous filler	½-in.	do	34½ to 45½	½	do	None	Nineteen, ¾-inch diameter, smooth, one end free.	do.
New Mexico	As shown on plans		do	30	¼ to ½	Prepared bituminous	When shown	do	
New York	Plain butt joint		do	40	½	do	None	do	
North Carolina	Not required		Construction	(1)			5 feet by ½-inch diameter	Eight or nine, 4 feet by ¾-inch diameter, one end free.	Half - width construction, see Figure 4.
North Dakota	Deformed metal plate	(3)	Expansion	30	½	do	5 feet by ½-inch diameter, deformed, 5 feet c. to c.	do	See sketch of drop curbs at pavement ends, Figure 4.
Ohio	do	18	Construction	(1)			3 feet by ½-inch diameter, 5 feet c. to c.	do	Heavy crown.
Oklahoma	do	18	Expansion	50	1	Poured bituminous	4 feet by ½-inch square, deformed, 3 feet c. to c.	do	
Oregon	do	18	do	25	¼	Prepared bituminous	4 feet by ½-inch diameter, deformed, 5 feet c. to c.	do	
Pennsylvania	do	14	do	(1)		do	2 feet by ½-inch square, 3 feet c. to c.	do	Expansion joint location pre-determined.
Rhode Island	Plain butt joint		do	100	½	do	4 feet by ½-inch diameter, 5 feet c. to c.	Eight, 2 feet by ½-inch diameter, smooth, one end free.	
South Carolina	Not required		do	40	¼ to ½	do	4 feet by ½-inch diameter, 5 feet c. to c.	do	
Tennessee	Deformed metal plate	18	do	30	¾	Prepared or poured bituminous.	4 feet by ½-inch diameter, 5 feet c. to c.	do	
Texas	do	18	do	60 to 100	¼ to ½	do	4 feet by ½-inch diameter, 5 feet c. to c.	Six, 4 feet or 5 feet by ½ or ¾-inch diameter, smooth, one end free.	
Utah	do	18	do	40	¼ to ½	do	4 feet by ½-inch square, 5 feet c. to c.	do	Tongue and groove construction joint.
Vermont	Poured bituminous filler	(3)	do	43½	½	do	2 feet by ¾-inch diameter, deformed, 3½ feet c. to c.	Eight, 2 feet by ¾-inch diameter, smooth, one end free.	
Virginia	Not required		Construction	(1)			2 feet by ½-inch square, smooth, 1½ feet c. to c.	do	See Figure 4 for sketch of contraction joint.
Washington	Poured bituminous filler	¼-in.	Expansion	60	¾ to ½	Poured bituminous	None	do	
West Virginia	Deformed metal plate	16	Construction	(1)			4 feet by ½-inch diameter, deformed, 4 feet c. to c.	Four, 4 feet by ½-inch diameter, one end free.	
Wisconsin	do	18	Expansion	31½	½	Prepared bituminous	do	do	

1 Four States, Montana, Nevada, South Dakota, and Wyoming, submitted insufficient mileage to justify inclusion in the table.

2 Paint coat.

3 Not shown.

4 Necessary.

5 Designed.

the large increase in cost. In general, the mixtures specified for concrete pavements range from 1:2:3 to 1:2:4, although certain States specify a cement-sand ratio of 1:1½ or 1:1¾ for special aggregates.

Less steel per square yard of pavement is now used as reinforcement than formerly, although the total amount of steel used, including edge bars, dowels, and metal joints is probably greater. Only seven States, six of which are located in the northern part of the country where the soil freezes to considerable depth, specify bar mats in preference to fabric reinforcement. The majority of States using a reinforced design specify fabric or permit either fabric or bar types to be used.

Figures 3 and 4 show sketches of some unique and interesting features of design found in certain States. In compiling the data for Tables 1 and 2 effort was made to eliminate, so far as possible, such designs as appeared to be unusual and to select the one most representative of the usual practice in each State. Certain States have developed standard designs which are apparently used without variation, while other States vary such features as depth of pavement, mix, amount and position of steel reinforcement, spacing of transverse joints or the shape of the cross section to fit local conditions. In four States so few projects involving concrete pavements have been received that no general idea of their practice in designing could be obtained.



HALF-WIDTH CONSTRUCTION OF CONCRETE PAVEMENTS AVOIDS DETOURING DURING CONSTRUCTION

Sand-clay and gravel surfacing.—There does not appear to be any material change in recent years in the technic of constructing sand-clay roads, but the tendency in clay-bound gravel construction is toward the use of smaller sizes of gravel in the surface course. Experience in maintaining gravel roads under automobile traffic has shown that gravel larger than 1 inch in size is loosened and "kicked out" of the surface and any considerable amount of large gravel is likely to result in failure of the surface by ravelling.

Bituminous macadam.—In the construction of bituminous penetration macadam there has been a notable trend toward the use of stiffer binder. As an illustration of this change, the penetration test requirements in Ohio have been reduced from a range of 90 to 120 in 1923 to 85 to 100 in 1926 and in North Carolina 90 to 120 in 1922 to 80 to 100 in 1926, and in New York from 120 to 150 in 1923 to 100 to 120 in 1926. There is also a tendency, in this type of construction, to use larger and more uniform sizes of crushed stone and to reduce the depth of the top course from the 3 and 3½ inches formerly used to 2 and 2½ inches.

The reduction in the depth of the course and the use of larger and more uniform sizes of stone permits more complete and uniform penetration of the course, and, together with the stiffer asphalt, tends to eliminate

the development of corrugations in the surface under heavy traffic.

Protection of the edges of bituminous macadam by an extension of the base course has been adopted to a



A CALIFORNIA ROAD, ORIGINALLY A 24-FOOT BITUMINOUS CONCRETE ON A CONCRETE BASE, WIDENED TO 42 FEET BY THE ADDITION OF CONCRETE SHOULDERS AND A NEW ASPHALTIC CONCRETE SURFACE APPLIED

limited extent and this feature is standard design in Ohio.

Bituminous concrete.—The effect of research and wider experience in the construction and maintenance of bituminous concrete pavements is shown in the tendency toward more rigid requirements in grading the mineral aggregates. The practice of constructing this type of surfacing on a rigid base of Portland cement concrete appears to be gaining in favor, due to the difficulty in properly consolidating new macadam bases and also to the small differential in the cost of construction.

Brick.—Until rather recently grout-filled brick on a sand cushion and monolithic brick pavements were standard construction, but these types are now practically obsolete. All of the Federal-aid brick pavement projects submitted for several years have provided for the use of a bituminous joint filler. The practice of laying bricks flat rather than on edge is now so commonly adopted as to be accepted as standard construction.



OLD PENETRATION MACADAM IN MARYLAND RECONSTRUCTED AND WIDENED BY THE ADDITION OF NEW CONCRETE SHOULDERS AND NEW PENETRATION MACADAM TOP

Recent test results indicate that brick as thin as 2½ inches may be used for paving. The report on these tests² has been issued only recently and it is anticipated that the results will be reflected in designs submitted in the future.

(Continued on p. 20)

² Thin Brick Pavement Studied, PUBLIC ROADS, vol. 7, No. 7, September, 1926.

PROPORTIONING CONCRETE AGGREGATES BY WEIGHT

By R. W. CRUM, Engineer of Materials and Tests, Iowa Highway Commission

IT HAS been demonstrated by numerous investigators that the strength of concrete depends upon the relation between the amount of cement in a unit volume and the total amount of solids in that volume. The weights of the materials are proportional to the volumes of solid material in the concrete, and it is therefore logical to express proportions by weight. Expressing proportions by weight also greatly facilitates computations of quantities of materials and the design of equivalent mixtures from dissimilar materials.

MEASUREMENT BY WEIGHING RESULTS IN ACCURATE PROPORTIONING

If the fine and coarse aggregates in concrete are to bear a definite relation to each other, the advantages of weighing the aggregates, instead of measuring them, are obvious. Weighing provides a means of securing accurate quantities of materials for each batch, without the necessity of taking into account the bulking effect of moisture as in loose volumetric measure, and without additional cost. The contractor as well as the State has an accurate record of the actual amount of material used. The concrete is more uniform in quality, and the inspector can more easily control the measuring and mixing operations and make changes in quantities when necessary. Experience in Iowa with some 40 installations gives very convincing evidence that the equipment needed is inexpensive, easily installed, and can be rapidly operated.

The primary objective in measuring the ingredients for concrete by any method of measurement is to maintain definite and constant relationships between the constituents of successive batches. Accurate volumetric measurement may be possible, but in actual practice even with first-class inspection, accuracy equal to that of weighing can not be attained without extreme vigilance and expenditure of time. It has been found easily possible to weigh consistently to 5 pounds or less. In an ordinary five-bag batch of 1:2:3½ concrete this means a possible error of 0.3 per cent. If measured by volume in a straight-sided box having an area of 6 square feet, a variation of 1 inch in height in filling the box means a possible error of 3.5 per cent.

WEIGHING AGGREGATE ADVANTAGEOUS FROM VIEWPOINT OF BOTH ENGINEER AND CONTRACTOR

The relative accuracy of the measuring method is, however, not the most vital factor. Even if the materials should be accurately measured by volume, changes in the moisture content of the sand might make relatively large changes in the batch. A large amount of research has been devoted to study of the bulking effect of moisture in sand and means of making corrections when measuring sand, but it is still common practice to ignore this factor where concrete is made in large quantities. The simplest solution is to weigh the materials and do away with the problem. The effect of moisture upon weight is very small as compared to its effect upon bulk, and the weight can be very quickly and easily corrected for the amount of moisture present.

Figure 1 shows the typical effect of moisture upon the bulk of sand. This effect is sufficient to produce an appreciable variation from theoretical proportions on practically every job where volumetric proportioning is used. Assuming that a 1:2:3½ mixture of dry materials measured by volume would require 1.42 barrels of cement per cubic yard to make concrete of 80 per cent density. If the sand used should contain 2 per cent of moisture, the cement content would be increased to 1.506 barrels per cubic yard—an increase of 6 per cent. Now if the materials

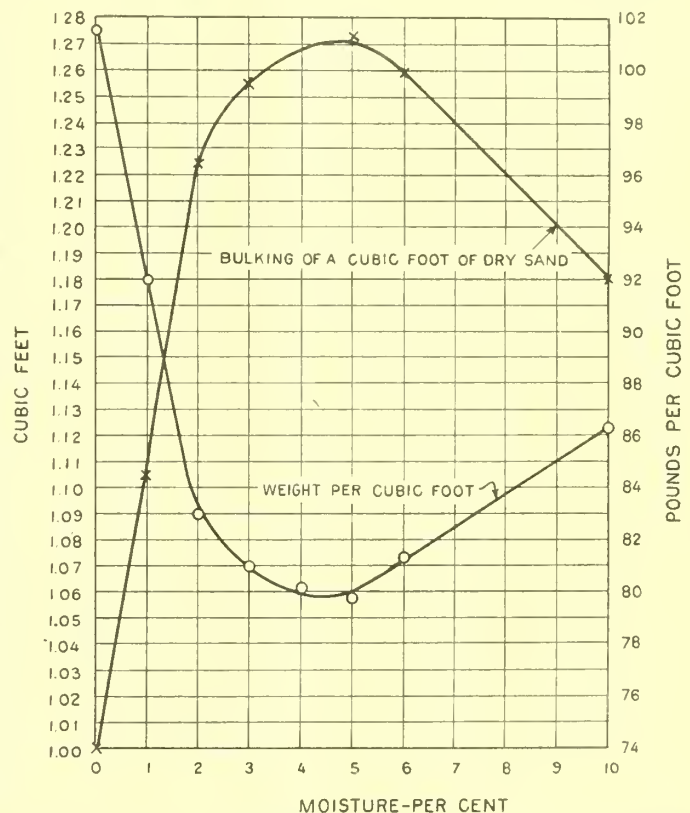


FIG. 1.—TYPICAL CURVES ILLUSTRATING EFFECT OF MOISTURE ON BULKING OF SAND

were to be weighed instead of measured by volume, the same result would be secured with dry materials, but with sand containing 2 per cent moisture the cubic yard of concrete would contain 1.43 barrels of cement, an increase of only 0.06 per cent. In other words, 2 per cent of moisture in the sand means an increase in the cost of cement, which is ten times larger when the materials are measured by volume than when they are measured by weight. Even this small increase could be lowered by correcting the batch weights for the moisture content.

Of course if proportions are based upon dry materials, any change in moisture with volumetric measurements will result in richer concrete, and hence the procedure may be safe, but there is a big advantage

in uniformity, and at least the contractor will recognize the advantage of not having the cement overrun the estimated quantity. The practice in Iowa is to run moisture determinations about six times a day or oftener and correct the batch weights accordingly.

One of the most serious criticisms of our concrete pavement work is the lack of uniformity in the quality of the concrete. There are, of course, a number of factors upon which the uniformity depends. Undoubtedly the accuracy of measurement of the various ingredients is one factor of major importance, and although improvement in this one thing will not entirely solve the problem, no solution can be had until the materials are measured with uniform accuracy.

CHANGE IN PRACTICE IN IOWA RESULTS IN MORE UNIFORM CONCRETE

The Iowa Highway Commission has for several years studied the composition of the concrete in the roads by making sieve analyses of samples taken once

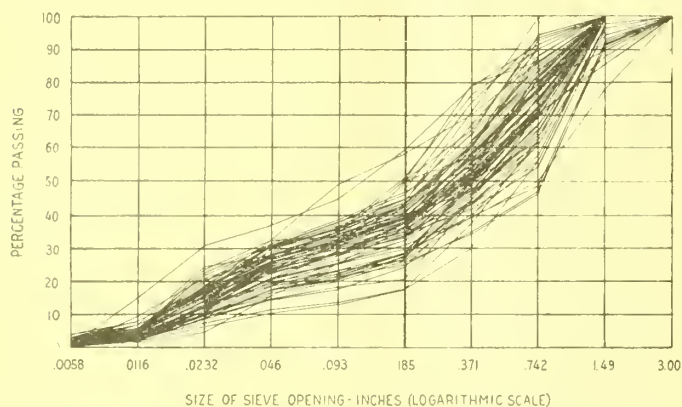


FIG. 2.—GRADING OF AGGREGATE ON A JOB WHERE VOLUMETRIC PROPORTIONING WAS USED. SAMPLES WERE TAKEN DAILY FROM THE CONCRETE AFTER PLACING AND THE CEMENT REMOVED BY WASHING

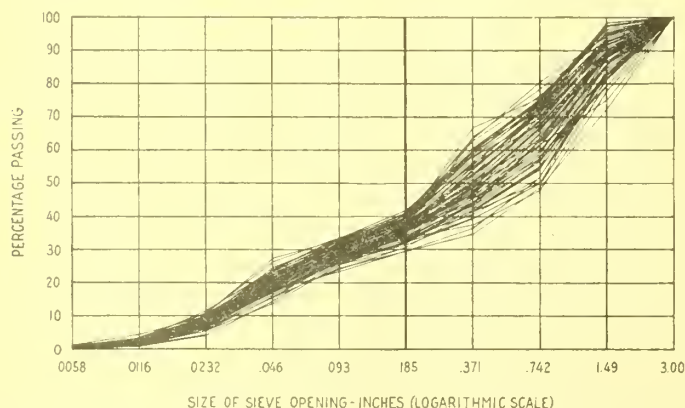


FIG. 3.—GRADING OF AGGREGATE ON A JOB WHERE MATERIALS WERE PROPORTIONED BY WEIGHT. SAMPLES WERE TAKEN DAILY FROM THE CONCRETE AFTER PLACING AND THE CEMENT REMOVED BY WASHING

each day from the road while yet wet. The cement is removed by washing and the aggregate remaining is dried and screened. Figure 2 shows these sieve analyses plotted for each day's run through a paving operation in 1921, and is typical of results secured by volumetric measurements, under first-class inspection.

According to the specifications, the mixture should have contained 34 per cent of fine aggregate. The

fine aggregate or sand is that part passing the No. 4 sieve having 0.185 inch square openings. As shown by the diagram (fig. 2) the proportions of the sand in the mixes actually passing the No. 4 sieve ranged from 18 to 60 per cent. Figure 3 is typical of results obtained since weighing was introduced in 1924. It will be observed that the sand content ranges only

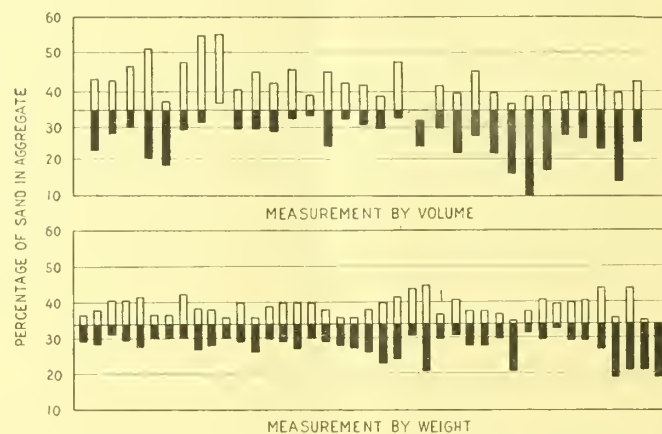


FIG. 4.—MAXIMUM AND MINIMUM PERCENTAGE OF SAND IN TOTAL AGGREGATE DURING A SEASON'S RUN ON A NUMBER OF JOBS SOME OF WHICH PROPORTIONED BY VOLUME (PRIOR TO 1924) AND SOME BY WEIGHT. SAMPLES WERE TAKEN DAILY AND EACH BAR REPRESENTS THE EXTREME VARIATION ON ONE JOB. IN ALL CASES THE SPECIFICATION REQUIRED APPROXIMATELY 34 PER CENT SAND IN THE AGGREGATE



FIG. 5.—VARIATION IN USE OF CEMENT FROM THEORETICAL REQUIREMENT ON JOBS PROPORTIONING BY VOLUME (PRIOR TO 1924) AND BY WEIGHT. EACH BAR REPRESENTS A SEASON'S WORK WITH ONE MIXER

from 30 to 41 per cent. This is not perfect, but is undoubtedly an improvement. Figure 4 shows the maximum and minimum sand content of concrete during a season's work on a number of jobs, some of which proportioned by volume, and others by weighing.

Proportioning by weight rather than volume avoids errors due to bulking and permits of easy correction for weight of moisture and thus reduces overrun in cement and aggregates due to these factors—or, in other words, makes the yield more uniform. The estimated quantities of cement and aggregates are usually based upon dry materials, and therefore, in measuring by volume, moisture, especially in the sand, will increase the cement per cubic yard and decrease the yield. In weighing, however, accurate batches can be produced and the yield per barrel of cement kept more constant. Our experience is as follows:

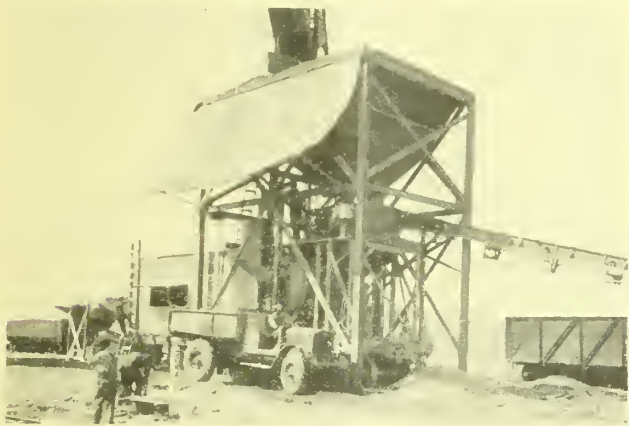
Out of 41 contracts during the period from 1920 to 1923, with aggregates measured by volume, the maximum average overrun in cement for a season's work

on one job was 10.8 per cent, the greatest underrun was 1.8 per cent, and the average for all jobs was 2 per cent overrun. Out of 23 contracts during 1924 and 1925, with aggregates weighed, the maximum overrun in cement was 3.6 per cent, the minimum 0.3 per cent, and the average for all jobs was 1.8 per cent. Variations in slab thickness will always cause variations in recorded cement content per unit of volume,

for sand and gravel and freight at such a rate that every cubic yard of concrete he made would have contained 3,900 pounds of sand and gravel. In the proportions specified, this would have meant the production of concrete of 105 per cent density. The producer of the material recognized this absurdity and settled for a more reasonable amount, but this settlement was on a guess and not based on facts as it would have been had the material been weighed when mixed.

STRENGTH OF CONCRETE PROPORTIONED BY TWO METHODS INVESTIGATED

There is no question but that the realization of the foregoing contractual advantages will result in closer bidding and reduction in cost to the public. Any reductions in construction uncertainties are bound to



BOTTOM DUMPING HOPPER SUPPORTED BY WEIGHING DEVICE USED AT A CENTRAL MIXING PLANT

and under rigid inspection the tendency will be to overrun. Data for 1926 are now shown for the reason that on account of the extraordinarily wet season, most of the contracts showed an unusually large overrun in materials, due to great difficulty in preparing subgrade. It would appear from the above data, which are shown graphically in Figure 5, that some improvement in uniformity was made.

A valuable result of the weighing of materials is the accurate record of the amount of material actually

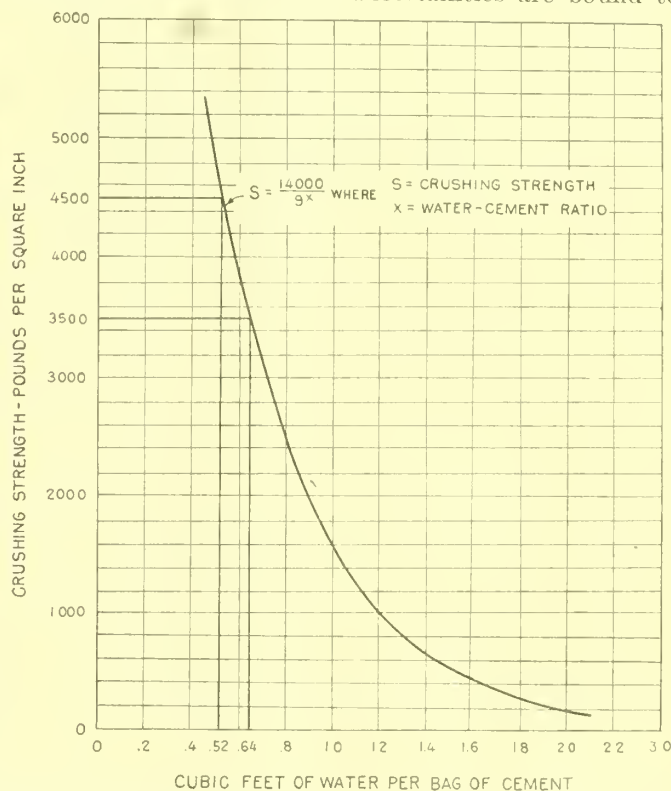


FIG. 7.—TYPICAL RELATION BETWEEN WATER-CEMENT RATIO AND CRUSHING STRENGTH OF CONCRETE. AN INCREASE OF 0.12 CUBIC FOOT OR 0.9 GALLON OF WATER PER BAG OF CEMENT LOWERS THE STRENGTH OF THE CONCRETE WITH A STRENGTH OF 4,500 POUNDS PER SQUARE INCH TO 3,500 POUNDS PER SQUARE INCH

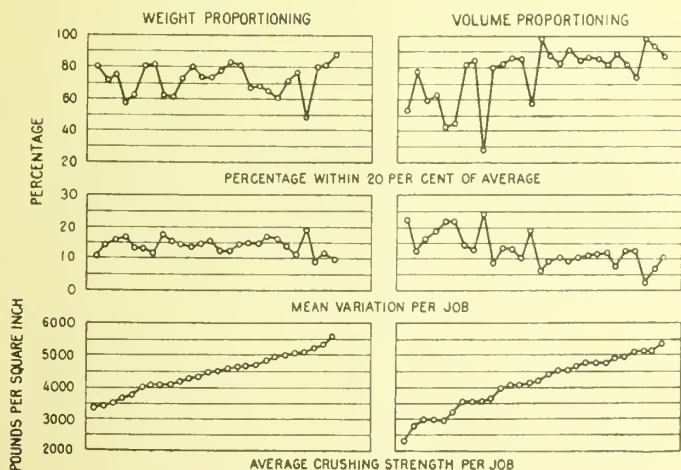


FIG. 6.—RESULTS OF COMPRESSION TESTS OF CYLINDERS MADE DAILY ON A NUMBER OF JOBS SOME OF WHICH PROPORTIONED BY WEIGHT AND SOME BY VOLUME. EACH POINT PLOTTED IS BASED UPON A SEASON'S WORK WITH ONE MIXER

used in the structure which is obtained. Since such materials are customarily bought and sold by weight, such a record should be of great value, especially to the contractor. At several times in the past few years settlements between the highway commission, contractors, material producers, and carriers would have been greatly facilitated had such a record been available. In one case, a paving contractor was billed

have this effect. The method is, therefore, justified without reference to the quality of the concrete. However, since it must be admitted that accurate and uniform measurement of materials is the basis of uniform concrete, some improvement in uniformity of concrete by means of weight measurements should result. Examination of the tests of cylinders made from day to day upon the paving jobs indicates this improvement to a limited extent only. The same range of strengths in individual specimens appears on the weighed-aggregate jobs as on those in which the aggregates were measured by volume. However, comparison of the average strengths of all of the different jobs from 1921 to 1926 indicates that the job averages on weighing jobs are more uniform. This conclusion is supported by the data shown in Figure 6. Also the percentage of specimens from the various jobs falling

within 20 per cent of the average strength appears to be the more uniform for the weighed aggregate jobs.

It would appear from study of these data that there must be some other variation in measurement that affects the uniformity of the strength. With the cement and aggregates accounted for, it is obvious that in order to secure the full benefit of the improved methods described, refinement must be made in the control of the water content of the batches. Data from many sources have established that the relation between water, cement, and strength may be expressed by a curve of the form shown in Figure 7. Since concrete of comparatively high strength is used on con-

system for three years. This was recently expressed at a meeting of the paving section of the central branch of the Associated General Contractors of America. The weighing of aggregates has been standard practice in Iowa for three years and is far past the experimental stage.

Several varieties of weighing apparatus have been successfully used. Simple field-built bins and hoppers used with ordinary platform or mill scales have been used as well as permanent equipment designed and built by manufacturers of paving equipment.

Four general types are in use, as follows:

1. Bottom-dumping hopper on lever-weighing system with direct reading dial scale.
2. Batcher device adapted to weighing system, usually with pipe lever scales.
3. Bottom or side dumping hopper on platform scales.
4. Hopper mounted on movable scales for loading industrial railway boxes.

In the experience of the Iowa Highway Commission those installations have been found most satisfactory which are so arranged that the scale beam and operator are at the same elevation as the weighing hopper, so that the operator can observe the inside of the hopper and easily remove excess material if necessary.

The operation of a weighing plant, of course, involves thorough inspection. It is the inspector's duty to make sure that the scales are always in proper working order, to calibrate them frequently, and to see



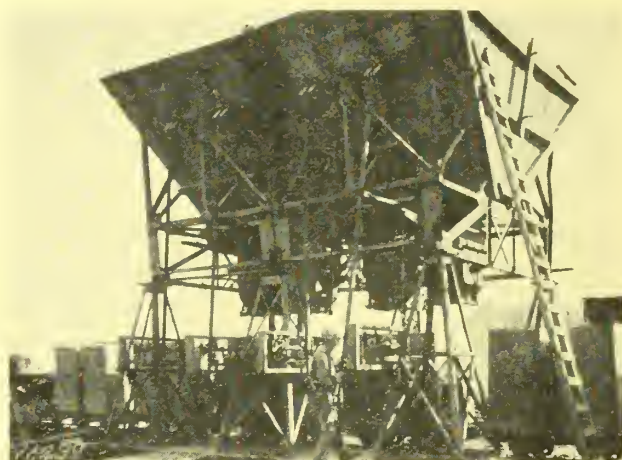
BOTTOM DUMPING HOPPERS BENEATH BIN SUPPORTED BY PLATFORM SCALES

crete paving work, reference to the curve for strengths from 3,000 pounds per square inch up will demonstrate forcibly that on such work the effect upon strength of a comparatively small variation in water content is very large.

Special investigations were made on two jobs where the aggregates were accurately weighed and the water-measuring devices were considerably better than ordinary, with the rather astonishing result that we found a variation in water-cement ratio in fresh concrete of from 0.684 to 1.072 cubic foot of water per bag of cement in one case and from 0.648 to 0.959 cubic foot per bag in the other. According to data published by Abrams, Talbot, and others, this variation in water-cement ratio would easily account for variation in strength of 100 per cent. The mixer operator and inspector were trying to maintain a uniform consistency while the observations were made. About seven samples per day were taken for three days. The engineer who made this investigation reported that carelessness in handling cement was a considerable factor in this variation. It is evident that more accurate methods for controlling the water-cement ratio must be developed, if we are to realize the maximum benefits from other refinements such as weight proportioning.

WEIGHING FOUND TO BE A PRACTICAL METHOD

There have proven to be no practical difficulties in handling aggregates by weight. The equipment cost is negligible and the time element is not a factor. The rate of production has not been influenced by this method of weighing. The best evidence of the advantage to the contractor is the unanimous approval given by the contractors who have worked under this



BATCHER DEVICE WHERE MATERIALS ARE WEIGHED BY AN ARRANGEMENT OF PIPES AND LEVERS

that the correct amount of material goes into each batch. The latter point involves frequent determinations of the moisture content of the aggregate and making the necessary corrections.

It would seem, since the weighing method provides a more accurate means of securing the specified amounts of materials in every batch, thereby removing one obstacle to the production of more uniform concrete, and since this can be done without extra cost, that we are justified in requiring this procedure. In the opinion of the Iowa Highway Commission this procedure marks a distinct advance in the technic of constructing concrete pavements or bases for other types of pavement.

CONCRETE BRIDGE TO BE TESTED TO DESTRUCTION

An unusual opportunity to test to destruction a large concrete bridge of modern design has been presented in North Carolina. A dam is now under construction on the Yadkin River which will cause the impounded water to entirely submerge the bridge across the river between Albemarle and Mount Gilead.

This bridge was built by the State in 1922 as a Federal-aid project and consists of three open-spandrel arch spans of about 150 feet each and 14 T-beam or deck-girder spans of approximately 40 feet each. The over-all length from face to face of the ends bents is 1,069 feet.

A cooperative agreement has been entered into between the North Carolina State Highway Commission and the Bureau of Public Roads for the making of as complete tests as possible before the bridge is submerged. It is believed that the results may be of great value to the engineering profession and in order to secure the best technical direction possible several organizations have been invited to appoint a representative on an advisory committee to take charge of the direction of the tests. An invitation has been extended to the following organizations:

- University of North Carolina.
- North Carolina State College.
- American Association of State Highway Officials.
- American Society of Civil Engineers.
- American Railway Engineering Association.
- American Concrete Institute.
- Highway Research Board.
- Bureau of Standards.
- American Society for Testing Materials.

At the present time all but two of these organizations have accepted the invitation. It is proposed that the advisory committee will formulate the test program and assume direction of the work from the technical standpoint. The program will be financed and carried out by the North Carolina Highway Commission and the Bureau of Public Roads.



YADKIN RIVER BRIDGE

The period during which the loading tests may be carried on will be limited to the interval between the time when it will be possible to close the bridge to traffic and the time when the bridge will be submerged by the water impounded by the dam. With this time limitation in view it will be necessary to make very complete preliminary arrangements for the work.

DISPOSITION OF MOTOR-VEHICLE FEES AND GASOLINE TAXES IN IDAHO AND UTAH

Attention has been called to the need for a more complete statement of the disposition of motor-vehicle and gasoline-tax revenues in Idaho than that contained in the article "The collection and disposition of motor-vehicle revenues" in the January issue of Public Roads.

The county assessors who collect the license fees are directed to remit 10 per cent of their collections to the State treasurer, who places these payments in the State highway fund to be used for the maintenance of State highways only. Ninety per cent of the fees collected accrue to the counties. Collection costs are paid out of funds appropriated for the department of law enforcement, and therefore the gross collections are divided between the State and counties.

All of the revenue from the gasoline tax is placed to the credit of the State highway fund from which the costs of administering the law are paid. The net proceeds may be used for either maintenance or construction at the discretion of the Commissioner of Public Works. No part of the gasoline-tax revenue accrues to the counties.

In the article referred to Idaho was incorrectly included in the group of States which distribute the gasoline tax and the license fees in the same manner. In this State the counties receive 90 per cent of the license fees and the State 10 per cent and the entire net proceeds of the gasoline tax accrues to the State.

In the same article it was stated that the gasoline-tax receipts in Utah are placed to the credit of a fund for payment of interest and sinking fund charges on State road bonds. This statement was incomplete, as all ex-

cess revenues are placed to the credit of the highway fund for the construction and maintenance of roads. In 1926 the highway fund derived \$1,000,000 from this source.

BOND OF PILE HEADS IN CONCRETE FOUNDATIONS TO BE TESTED

The Bureau of Public Roads is planning a series of tests to determine the bond strength developed between the heads of foundation piles and concrete foundation seals and to investigate methods of anchoring reinforcing steel to pile heads. It is planned to make these tests at an early date in order to secure information for the design of Federal-aid bridges which have been proposed for the Gulf region. In the design of these bridges special attention is to be given to the pier designs in order that they may be capable of withstanding large overturning moments due to high winds.

Six series of tests are to be made at the Arlington experiment station of the bureau. Short sections of water-soaked timber piles will be incased in blocks of 1:2:4 concrete representing the foundation seal. Plain cylindrical piles and pile heads which have been expanded by means of wooden wedges are to be used and will be pulled from the concrete by means of hydraulic jacks.

The relative effectiveness of different methods of anchoring reinforcing steel to pile heads is also to be determined. These tests will include hacked bars driven into pile heads, bars with fox-bolt ends driven into pile heads, and special steel pile rings designed for the attachment of reinforcing bars. The anchorage developed will be determined by pulling the steel from the pile head in a large testing machine.

(Continued from p. 6)

One section of the Gansevoort road in New York shows no defects while an adjoining section is a total failure, the difference again being due to change in subgrade.

W. F. Purrington, of the New Hampshire Highway Department, points out that a section of gravelly soil road treated with bituminous materials has now accommodated traffic through two winters, while wheels sank to their hubs where the road was not treated. It is plainly indicated that additional support is afforded by this simple treatment which may warrant a reduction in the thickness of gravel, macadam, or bituminous roads laid on such surfaces.

East of Cleveland, Ohio, brick surfaces laid on 4-inch concrete base have been in service for years, while a brick road laid on a 6-inch concrete base south of Cleveland was not adequate. In this case, the difference in subgrade support had a value of several thousand dollars per mile.

In a survey of 1,277 miles of California roads in 1920 it was found that no failure occurred on sand subgrade, 24 per cent of the failures occurred on loams and 76 per cent occurred on adobe and clay subgrades.

The State of Delaware has found concrete with a 5-7-5 cross section more satisfactory in some locations than concrete with a 6-8-6 cross section in others. The difference in support in these instances has had a money value of at least \$3,000 per mile.

Some of these examples may be considered as extreme, and in some cases variables other than support may be factors, but they serve at least to indicate the possible economic savings which may result from the intelligent suiting of pavement design to subgrade condition.

(Continued from p. 14)

Methods of reconstruction.—The reconstruction of old pavements which have become badly worn or are of inadequate depth and width for present traffic is a problem of considerable magnitude in some of the States. In the majority of cases the old surfacing has

considerable value and the method of reconstruction adopted should conserve this value as far as possible. The method of widening old, narrow macadam surfaces adopted by the State Roads Commission of Maryland has been used on a large mileage in the State. At the edges of the old macadam surface, concrete shoulders are constructed. These are generally 2 feet in width and 8 inches in depth and are placed at an elevation which will permit resurfacing the existing macadam and at the same time eliminate the excessive crown found on many of the old roads. Resurfacing is done with either bituminous macadam or bituminous concrete. Excellent results are being obtained by this method at a moderate cost per mile. The same plan is being followed in other States and in some cases the side strips are made sufficiently wide to form traffic lanes. The resulting pavement is particularly pleasing to fast traffic because of the definite and ample separation of traffic.

Old concrete pavements have been successfully widened and resurfaced by constructing an additional layer of plain or reinforced concrete over the old pavement and extending the new slab beyond the edges of the old pavement. In such cases the new portion of the pavement is usually not less than 5 inches in depth at the center and thickened at the edges to equal the combined depth of old and new pavement. Contrary to expectation, such pavements have not shown any marked tendency to develop cracks along the edges of the old pavement or over cracks in the original surface. Old brick and bituminous pavements on concrete bases, as well as old macadam surfaces, have also been successfully resurfaced and widened by the same method.

Concrete roads have also been resurfaced with bituminous concrete, placing a first course which takes out the inequalities of the old surface and then the wearing course. Where additional width has been desired this has been secured in some cases by placing new concrete shoulders and in other cases by laying a strip of bituminous concrete base on each side of the old pavement and laying the surface course on both the new and the old construction.

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Report of the Chief of the Bureau of Public Roads, 1925.
*Report of the Chief of the Bureau of Public Roads, 1926. 5c.

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Vol. 5, No. 19, D- 3. Relation between Properties of Hardness and Toughness of Road-Building Rock.
Vol. 5, No. 24, D- 6. A New Penetration Needle for Use in Testing Bituminous Materials.
Vol. 10, No. 5, D-12. Influence of Grading on the Value of Fine Aggregate Used in Portland Cement Concrete Road Construction.
Vol. 11, No. 10, D-15. Tests of a Large-Sized Reinforced-Concrete Slab Subjected to Eccentric Concentrated Loads.

* Department supply exhausted.

FEBRUARY 28. 1927

* Includes projects reported completed (final vouchers not yet paid) totaling: Estimated cost \$ 94 188 742.31 Federal aid \$ 40 222 318.15 Miles 3 712.5

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A JOURNAL OF HIGHWAY RESEARCH



UNITED STATES DEPARTMENT OF AGRICULTURE
BUREAU OF PUBLIC ROADS



VOL. 8, NO. 2



APRIL, 1927



A TYPE OF LANDSLIDE COMMON IN WEST VIRGINIA

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U. S. DEPARTMENT OF AGRICULTURE

BUREAU OF PUBLIC ROADS

CERTIFICATE: By direction of the Secretary of Agriculture, the matter contained herein is published as administrative information and is required for the proper transaction of the public business

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R. E. ROYALL, Editor

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LANDSLIDES AND THEIR RELATION TO HIGHWAYS

A REPORT OF OBSERVATIONS MADE IN WEST VIRGINIA AND OHIO TO DETERMINE THE CAUSE OF SLIDES AND DEVISE MEANS OF CONTROL

Reported by GEORGE E. LADD, Associate Economic Geologist, United States Bureau of Public Roads

PART I

This, the first of two articles which will deal with the causes and control of landslides in West Virginia and Ohio, is devoted to the geological characteristics of the region and the causes of slides. A subsequent article will deal more particularly with the engineering problems involved and the various methods of control which have been tried, illustrated by reference to a number of specific cases of slides. Physical analyses of materials and the results of laboratory soil tests might properly be included in this first article, but their presentation has been deferred in expectation that it will be possible to treat this phase of the subject more adequately after the completion of work now in progress.

THE geological formations and topography of a large part of West Virginia and Ohio are responsible for two sets of economic conditions—one good, the other bad, of which, fortunately, the good far outweigh the bad. The formations themselves contain abundant supplies of coal, oil, and natural gas, and of clays and shales suitable for the manufacture of a great variety of products. The peculiar topography

solve result from landslides for the frequent occurrence of which the geological formations and considerable annual rainfall are responsible.

Such slides cost these States annually a million dollars or more in extra construction work and maintenance of roads. (Figs. 1 and 2.) They also cause great annual expense to railroads and, in a lesser degree, to power, telephone, telegraph, gas, oil, and water-supply companies. Occasionally they damage a residence or a cemetery, and to some extent farm and grazing land.

Roads are affected in various ways. The slides may merely fall upon them and leave several thousand cubic yards of detritus to be removed. They may be dislocated or thrown out of line by the impact of the sliding material. The whole mass of material above, immediately beneath, and beyond and below the road may move, covering, dislocating, breaking and upheaving, or sinking the road. Any one or more of these, which may be called natural movements, may take place at a single locality. Yet, in their combined effects, these movements of material from a natural



FIG. 1.—DAMAGE TO A HIGHWAY BY A LANDSLIDE RESULTING IN SUBSIDENCE, UPHEAVAL, AND LATERAL DISLOCATION

developed in them furnishes an immense potential water supply and great reaches of navigable streams. Together these constitute a tremendous natural endowment of great value and economic importance.

LANDSLIDES A SERIOUS ROAD PROBLEM

To the road builder, however, these same formations and the resulting topography present a problem which, not yet solved, makes road construction and maintenance both difficult and expensive, particularly in West Virginia. In the latter State valleys are narrow and deep and there are no compensating extended ridges. Roads are, therefore, confined to the valleys in many of which the best locations are already occupied by railroads. As a result the roads are necessarily winding and nearly always up and down hill, road mileage between points is increased and construction is difficult. But the most serious of the obstacles which nature has placed in the path of the road builder to



FIG. 2.—IN SOME LOCATIONS THERE IS A CONTINUOUS FALL OF MATERIAL UPON THE HIGHWAY AND IT IS PARTICULARLY HEAVY IN THE WINTER AND SPRING MONTHS

position are not more destructive, perhaps less so, than the movements of artificially placed fill material which carry away the shoulders of the roads and undermine the pavement back to the solid shale or sandstone. Conditions under which such movements occur are illustrated in Figure 3.

GEOLOGY OF AREA STUDIED

The area affected by slides, as shown on the map, Figure 4, includes the eastern and southeastern portions of Ohio where very many occur, and, roughly, the western half of West Virginia, with local regions, lying mostly in Mineral and Grant Counties, where slides are frequent. Slides also occur in scattered localities throughout the southern tier of counties in Ohio.

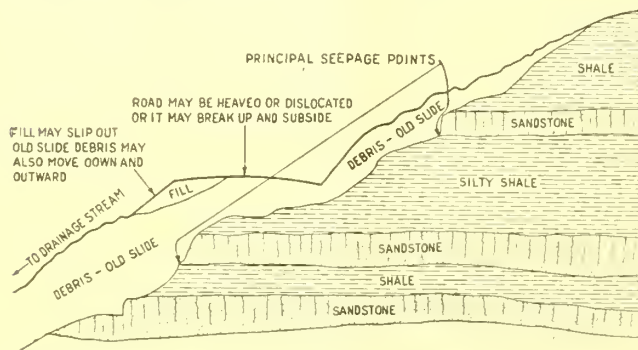


FIG. 3.—DIAGRAM SHOWING MANNER OF ACCUMULATION OF DETRITUS, SUCCESSION OF SANDSTONE AND SHALE BEDS AND NATURAL SEEPAGE PLANES. THE ROAD RESTS ENTIRELY ON MATERIAL WHICH IS NOT PERMANENTLY STABLE

The area lies at the southern limit of, and almost entirely beyond the influence of the ice invasions of the glacial period in a great geosynclinal trough or elongated bowl, the axis of which begins in Greene County, Pa., and extends southwesterly into Kentucky. In this great trough the Pennsylvanian or Upper Carboniferous formations have been preserved from erosion. These formations include the Monongahela, Conemaugh,



FIG. 4.—OUTLINE MAP SHOWING BY SHADED PORTION THE AREA IN WHICH LANDSLIDES ARE OF COMMON OCCURRENCE

Pottsville, and Allegheny series, and it is in these, particularly in the Monongahela and Conemaugh, that a large percentage of the slides in both States occur. To a much less extent the base of the Permian (Dunkard) formations also furnishes materials for slides, and along the southern border of western Ohio occasional slides occur in the Niagara series of Silurian age, and the Maysville of Ordovician age. The latter are con-



FIG. 5.—TYPICAL BED OF SHALE WITH INTERCALATED LAYERS OF SANDSTONE

finer to localities where very pure, "fat" clay shales have had an opportunity to become water-soaked.

Within this area where landslides are so common the geological formations consist of alternating strata of sandstones and shales, with occasional beds of hard, compact, so-called fire clay, and, in some localities, thin beds of siliceous limestone. Associated with these are numerous beds of coal.

The sandstone strata are traversed by numerous joint planes or fractures. They vary greatly in continuity, and rapidly in thickness. Frequently, and often abruptly, they become finer in grain until there is complete transition into shale. As a rule, the grains are not thoroughly cemented together and generally they exhibit a lack of toughness which makes them unsuitable for use as a road material.

The shales particularly in West Virginia are characteristically siliceous, even the purest of them containing enough silt to make them slightly gritty. They also contain varying amounts of lime and iron, often a great deal of both. As a general rule the beds are more massive and less broken than the sandstone strata, though they commonly contain thin layers of sandstone. The sandstones, on the contrary, are often split up into many thin beds separated by shale seams. (Figs. 5 and 6.)

Lying within the great geosynclinal trough previously referred to, these alternating beds of shale and sandstone dip generally toward the axis of the trough, but the major syncline is broken by minor folds so that there are numerous local anticlines and synclines.

In the portion of the area which lies in West Virginia the hills range from 100 to 2,300 feet in height. All are characterized by marked sharpness of slope. In eastern Ohio, except in the territory immediately adjacent to the Ohio River, the hills are lower and more gently rounded, and the deep narrow valleys of western West Virginia are largely absent.

These conditions, the nature and composition of the two principal rock materials; the vertical joints in the sandstone layers; the massive and impervious character of most of the shales; the dip of the strata (even though very slight); the characteristic topography of the region; and the abundant rainfall, averaging 43 inches a year; all have a direct bearing on the occurrence and character of the landslides in the two States, of which there are four principal types involving material in its original or natural position.

LANDSLIDES CLASSIFIED

The great majority of the slides are flow movements, either creeping or sudden, of the detritus which mantles the solid rock of the hillsides, and which results mainly from the weathering of the exposed shale surfaces.



FIG. 6.—BED OF SANDSTONE BEING UNDERMINED BY WEATHERING OF UNDERLYING SHALE. NOTE THE VERTICAL JOINT IN THE SANDSTONE

These are extraordinarily frequent and involve relatively large masses of material. (Fig. 7.)

Second in number, perhaps, and in the seriousness of their effect upon the highways, are those in which superficial weathered rock is precipitated as a matter of slope adjustment, as illustrated in Figure 2. A third class of slide consists of comparatively small-scale adjustments of jointed material where a vertical face results from artificial cut or stream erosion. (Fig. 16.)

Least frequent of all is a fourth class of slide in which the material moved consists of large masses of solid rock. Only one slide of this type has been brought to the writer's attention in Ohio,¹ and only a few have occurred in recent times in West Virginia. The most important of these occurred in Mineral County just south of Cumberland, Md., where a large rock mass



FIG. 7.—A NATURAL LANDSLIDE IN WEST VIRGINIA. THIS IS THE BEGINNING OF A TOPOGRAPHIC BREAK THROUGH THE CREST OF A HILL

went out as a result of a highway cut in stratified rock which dipped sharply toward the road. (Fig. 17.)

In addition to the foregoing classes, all of which involve material in its original or natural position, there is a fifth class which involves the movement of artificially placed highway fill material. Probably more than 50 per cent of the damage to roads and the most serious highway problems in the two States result from slides of this type. (Fig. 22.)

It is apparent from the mere description of these types of slides that they are caused by different combinations of the several geological and topographic conditions previously mentioned, and that no single method will suffice to prevent them. Preventive measures in each case must take account of the manner in which the slide is caused. As a basis for subsequent discussion of possible preventive measures, it will be desirable now to investigate the manner in which each of the classes of slides is caused and determine, in each case, the effect of the several contributing geological and topographic factors.

DETRITAL MASSES LUBRICATED BY CLAY AND ROUNDED SAND

The class of slides which has been described as a flow movement of detrital masses—the predominant type in the area studied—is conditioned by the nature of the material composing the mass and the geologic structure upon which it rests. The detritus consists of sandy clay originating from shale beds and of sand and sandstone fragments and boulders originating from massive beds of sandstone or from the intercalated sandstone layers often scattered through the shale beds. The sandstones are sufficiently hard strongly to resist

¹ This slide has not been visited by the writer but is quite likely a case of the undermining of a sandstone stratum by weathering influences.

weathering influences, and the shales when continually moist are very resistant to simple erosive influences although they yield to weathering processes because of their predominant clay content. The swelling of the clay when wet and shrinking on drying lead to cracking, curling, and peeling off of thin layers where the shale is exposed to weathering.



FIG. 8.—CHARACTER OF MATERIAL WHICH COMPOSES LANDSLIDE DETRITUS. NOTE ABUNDANCE AND COARSENESS OF SANDSTONE FRAGMENTS

Where the shale-clay is very silty, that is, full of very fine sand or silica particles, ordinary physical disintegration occurs rapidly. Under existing weather conditions such surfaces are alternately wet and dry and there is a continual sloughing off of the exposed shale face. The fine material thus produced falls and accumulates on the slope below. Rain and wind remove this material to some extent and deposit it further down the hill. In this way the shale face recedes as long as it is exposed to weathering influences and consequently it undermines its overlying bed of sandstone. In time the undermined sandstone breaks off and falls, often as enormous boulders which are subsequently broken up into smaller fragments by freezing of water within minute fractures which penetrate the sandstone mass. The cover-page illustration shows an enormous block of sandstone broken loose from its bed as the result of undermining through the weathering of an underlying shale. It illustrates one of the stages in the production of the mixed detritus which covers the hillsides of this region.

Figure 6 illustrates the retreat of a shale bed and the undermining of its overlying sandstone. In this case the laminations of the sandstone are such as to produce small fragments on falling. Note the vertical joint which permits circulation of water through the sandstone bed and that such water must escape along the top of the underlying, impervious shale bed.

The result of these processes is the accumulation of detrital masses consisting of various mixtures as described above. The proportions of clay, silt, sand, and sandstone in these masses vary considerably, but some clay is practically always present and when wet acts as a lubricant within the mass and is an important factor in producing slides.

On superficial examination many of these masses appear far removed in character from material that could easily become unstable and readily slump or slide. (Figs. 8 and 9.) Wherever surfaces are free from vegetation the material seems to be too rocky and sandy for such behavior, but such surfaces are deceptive. The sandstone fragments, varying from

small, angular pebbles to large boulders, which appear to form a large percentage of the mass, are merely a residue which remains after the removal of the surface clay by rain, and the interior of the mass in which clay remains is different in character.

The shales of West Virginia and the products they form in weathering are so full of silt and fine sand that they are perceptibly gritty, but in Ohio they are more clayey and the further west they occur the nearer do they approach to pure clay.

During the field study it was found that many masses of detritus slumped or slid in spite of the fact that they appeared to contain little clay matter and it was thought that the physical condition of the sand and silt particles might be a factor of importance. A microscopic examination of numerous samples was made and it was found that the sand and silt particles were highly rounded. Thus a quicksand condition was found which accounts in part for the ease of movement of some of these detrital masses when wet. Although the detritus is rich in sandstone fragments and boulders there still remain in it sufficient lubricating clay and rounded quartz particles to give it a degree of fluidity which leads to landslides even though the water content may be relatively low. It is probably true, especially in West Virginia, that many of the superficial masses



FIG. 9.—TYPICAL APPEARANCE OF LANDSLIDE DETRITUS

which slide would not do so if the large amount of contained sand and silt were sharp and angular.

We have then, on the hillsides of this area, masses of material which become unstable when the moisture content passes a certain critical point which of course depends on the composition of the mass and is therefore variable. The water content not only increases the

lubricating properties of the clay and rounded sand but also adds weight to the mass. The heavy annual rainfall is sufficient to supply considerable moisture by direct surface application but the underlying rock structure is such that it is also an important factor in supplying moisture to the detrital masses.

UNDERGROUND WATER A FACTOR IN CAUSING SLIDES

Reference has been made to the massive, impervious character of the shales, and to the vertical joints existing in the sandstones. (See Fig. 6.) These conditions are characteristic of the two materials in this region. During periods of folding the shales, being

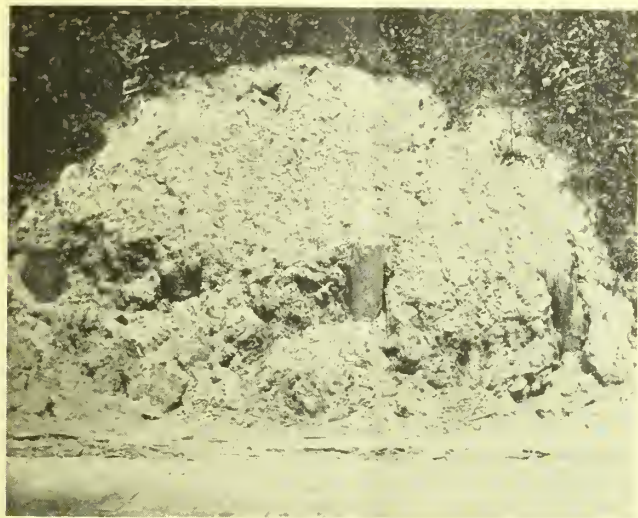


FIG. 10.—DETRITAL SLIDE MOVING THROUGH PILES AND SHOWING FLOW CHARACTERISTICS VERY PLAINLY. THIS SLIDE OCCURRED IN OHIO WHERE THE DETRITUS CONTAINS MORE CLAY AND LESS SANDSTONE THAN IN WEST VIRGINIA

somewhat flexible, have largely retained their massive character and mass continuity except in cases of violent disturbance. The sandstones being less flexible have been jointed. The result is that underground, permeating waters are able only here and there to penetrate the shale beds whereas they can flow with considerable ease along joint planes in the sandstone. The shale beds are not uniformly impervious to water however. Some of them are so interbedded with thin layers of sandstone and very silty clay that water can work through them vertically to some extent and to a considerable extent laterally. They are also slightly jointed, though very much less so than the sandstones. Occasionally they have been so broken and tilted by folding as to become somewhat pervious.

A condition is often found where water penetrates the mass of a high, broad ridge and finds its way freely through sandstone formations and passes along the upper surface of shale beds. It penetrates the shale beds to some extent through fracture planes but generally only in small quantities. If there are a number of alternating beds of sandstone and shale, only a small amount of water will reach the interior of the lower bed.

In many cases some surface water does penetrate great thicknesses of these alternating shales and sandstones but seepage of water is greatest along the upper surface of the shale beds. Water will naturally escape most readily where there is a slope to such a sur-

face. Therefore, seepage will be greatest on the hill-sides toward which the structural slope is directed. This means that water will saturate the superficial detritus on such hillsides more readily than on the opposite sides where the strata rise to the point of reaching the surface. It is believed that this is true even when the dip involved is very slight. This does not mean that slides will not occur on both sides of a ridge across which the strata may dip, but it does mean that they will be more frequent and more extensive on the down-dip side.

The great geosynclinal fold, which has been described, causes a general dip of the strata occurring in West Virginia in a direction a little north of west, and, in eastern Ohio, a corresponding dip averaging a little south of east. Minor folds cause local variations of these dips, some of them of considerable magnitude. The dip is sometimes steep but generally it is gentle and one can travel widely in many parts of the area and not notice that there is any dip whatever.

SIZE AND RATE OF MOVEMENT VARIES

Summarizing the foregoing paragraphs, it can be said that slides of detrital masses are brought about by the lubricating qualities of the clay and rounded sand content. The degree of instability depends upon the water content and conditions exist quite generally where this is supplied by underground seepage water, as well as surface water. The slope of the hillside upon



FIG. 11.—ANOTHER VIEW SHOWING THE FLUID NATURE OF THE OHIO DETRITUS

which a given mass rests and the conditions of support surrounding it are of course factors in determining the point at which a mass will begin to move. The various illustrations show that the topography is characteristically more or less steep but that cases have been found where a sufficient degree of fluidity has been reached to cause slides on gentle slopes.

A detrital mass may be practically continuous for 300 to 600 feet or more upward, but the slope may be intersected by broad benches of sandstone which until completely overridden tend to protect detritus below from the influence of upper slides, and to limit the



FIG. 12. VIEWS OF THE TOP, MIDDLE, AND BOTTOM OF A LANDSLIDE NEAR HUNTINGTON, W. VA., ARRANGED IN THE ORDER MENTIONED. NOTE THE BEGINNING OF A BREAK AND THE OLD SLIDE IN THE TOP PICTURE, THE FLUID CHARACTER OF MATERIAL IN THE CENTER PICTURE AND THE UPHEAVAL OF MATERIAL AT THE BOTTOM DUE TO IMPACT OF THE SLIDE

volume of a given slide. Vertical sections of detritus where thickness may be observed are rare, but as a result of many observations of the upper and lower portions of slides and cuts made by steam shovels in removing their débris from highways it appears that the average thickness is not more than 6 or 8 feet. Often over wide areas it is less. The maximum thickness appears to be about 12 feet except on gentle slopes where it may be somewhat thicker and in the lower extremities of old slide masses.

A great majority of individual slides involve a lateral extent varying from 150 to 300 feet; some are more extensive, others less so. Occasionally a long reach of detritus on the verge of instability as a whole may become involved in a local slide and within a few days from the first movement a mass up to a half-mile in length, measured along a road, may be in motion. Most of the natural slides studied measured in length up the slope 150 to 300 feet and in width when considered as individual slides from 200 to 400 feet. A few were found which were 1,000 feet or more broad. Such slides often have no break at the top but thin out there and thicken downward in successive waves which terminate in broad, gently curving lobes.

These slides seldom move more than 50 to 100 feet before a period of stability occurs, which may be merely days or may last months or years. Attention has been called to the fact that individual slides have been seen extending up the slope for a distance of 300 feet. In examining recent slides that have impinged on highways practically continuous slide material has been found for a distance of 600 feet or more up the hill. In such cases, however, the upper portions consist of older slides. Natural slides tend to begin near the top of hills or ridges and are later brought about successively below. The age of many of the older upper slides may be approximately determined in forested areas by the position and relative ages of trees. In one case an examination of old slides 400 feet vertically above a recent slide onto a highway disclosed growing on the rough, pitted slide-mass, one set of trees tilted in various directions and appearing to be from 100 to 125 years of age, while a younger growth all practically erect were approximately 60 years of age. The slide, therefore, occurred somewhere between 60 and 125 years ago but probably not much more than 60 years ago.

The slide movement is sometimes sudden and completed within a few minutes or hours. More often it is leisurely and a matter of days or weeks. Sometimes it consists of a series of short advances repeated at intervals throughout a period of years. Slides from the high, steep bluffs of the Ohio River sometimes drop like avalanches. Although a few of these involve thousands of yards the quantity of material in them is usually small. On high and excessively steep slopes detrital matter can not accumulate in great quantity because slides are too frequent and direct erosion constantly removes the finer material. Near the base of these bluffs there is frequently a wide bench or an area of gentle slope which permits accumulation of detritus there. In such cases when water saturation becomes sufficient large slides move upon the river road.

Such a mass accumulated on the road going up the Ohio River a few miles north of Huntington, W. Va. Old rough, pitted slides are present from 100 to 200 feet up the slope. Two years ago the bottom of this mass which had been cut through when the road was

graded moved against and upon the pavement. The road surface was broken and heaved for a distance of 200 or 300 feet, and several thousand yards of débris suddenly covered the road. Temporary stability was then attained, followed by renewals of the forward movement at varying speeds, but most of which were low.

This slide was inspected on the morning after a mass of detritus 12 feet in thickness had been cleaned from the road by a steam shovel. A thin wedge of detrital clay projecting across the ditch from the base of the landslide mass was drying out and had been lifted clear of the ditch. The position of its upper edge was marked on the margin of the pavement. Five hours later it was found that this edge had advanced over the pavement a distance of $2\frac{1}{2}$ inches. Meanwhile the sun had been shining on this thin edge



FIG. 13.—FISSURES AND POCKETS ARE TYPICAL OF DETRITAL SLIDES. THEY CATCH SURFACE WATER AND ARE OFTEN FACTORS IN KEEPING THE SLIDE MOVING

drying and shrinking it. Probably it had shrunk at least an inch laterally in the interval between observations so that the rate of movement of the landslide, at that particular time, was approximately $3\frac{1}{2}$ inches in five hours.

FLOW MOVEMENT OF MANY SLIDES OBVIOUS

There is a widespread opinion among engineers who have to deal with these slides that their movement is one of mass along a sloping glide-plane. They sometimes resort to drilling and blasting for the purpose of breaking up and roughening the surface underlying the slide débris as a mean of control. Observation of hundreds of slides of this type is convincing that movement is usually throughout the mass and not a movement of it as a more or less solid whole, and that such measures are seldom effective. This method of control will be discussed in more detail in a following paper.

The flow character of many slides is plainly obvious especially in Ohio, and occasionally in West Virginia. This type of movement is well illustrated in Figures 10 and 11. These slides consist of much clay and more or less highly rounded, small particles of sand and silt.



FIG. 14.—NATURAL LANDSLIDES MODIFYING TOPOGRAPHY. THE BENCHES ARE DUE TO HARD SANDSTONE STRATA



FIG. 15.—LANDSLIDES HAVE BEEN AN IMPORTANT FACTOR IN THE DEVELOPMENT OF TOPOGRAPHY IN WEST VIRGINIA. DOTTED LINE SHOWS OLD SKY LINE

The slide illustrated in Figure 10 is not satisfactorily held back by the row of piling placed for control because when wet the material becomes so fluid that it flows between piles. The mud-flow character of the slide illustrated in Figure 11 is obvious. The hill slope is probably not a factor in the movement of this mass which, although but 10 or 12 feet in thickness, can not support its own weight when saturated with water. In these cases the mass movement is better described as a slump rather than a slide.

The position of trees on recent slides throws light on the nature of the movement. The inclination of trees naturally depends upon the relative motion of the surface as compared with the material immediately beneath the surface. In the center of Figure 12, A, there may be seen a time-rounded mass of an ancient slide. Immediately in front of this is a tree of considerable age which had been tilted backward and subsequently developed for its upper part a vertical position. The recent slide in the foreground played no part in the position of this tree. A definite flow of semifluid

material is plainly shown in Figure 12 which shows views of the top, slope, and bottom of a single slide.

Fissures often 8 or 10 feet deep and in parallel rows at right angles to the direction of movement are common phenomena in slides where fluidity of the mass is not high. These fissures are gathering places for surface water and so tend to keep the mass moving. (Figs. 12 and 13.) It often happens that old detritus which is well compacted and sodded on the surface is squeezed and rolled up to a height of 7 or 8 feet by the impact of a slide from above (fig. 12, C) or added weight artificially or naturally imposed upon it.

TOPOGRAPHY DEVELOPED BY DETRITAL SLIDES

In going over this region the geologist is surprised at the major influence detrital slides have had on the formation of topography. Erosion gradually lowers the stream beds, but aside from this its action is confined almost solely to removing the products of landslides. Gulleys and ravines on hillsides are rare and are usually located upon a course prepared by landslides. The

whole process of the continuing development of topographic form, or reducing continuous ridges to short ones, or to isolated hills, is easily observed and strikingly demonstrated.

The development of topographic forms through the agency of landslides is illustrated by Figures 7, 14, and 15. In Figure 7 a natural landslide is shown, but it is foreshortened and the hill flattened as a result of photographic limitations. This slide starts over 300 feet up the slope of the hill and about 150 feet vertically above its toe. At its head lies a jointed bed of sandstone. It may be considered as the beginning of a broad concave cut in the hill which will ultimately reach and involve its crest. Note its glacierlike form, the evident flow of its material, the sharp break at its top, and the piled-up, thickened lower end.

Figure 14 shows a deforested ridge with characteristic benches which are in part if not wholly due to the presence of relatively hard sandstone strata. On the left end of the main ridge two natural slides appear, the upper one being the more recent. Evidence of other slides may be seen at various points toward the right at high elevations. The lower mass of this

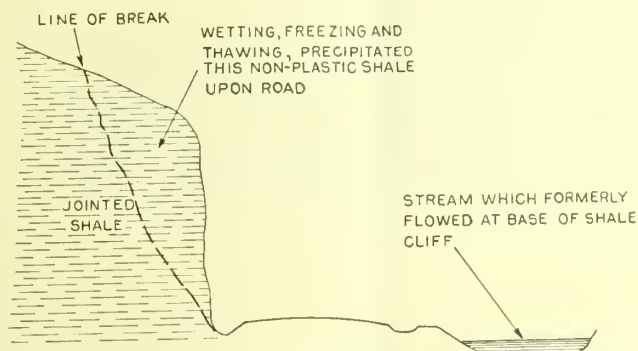


FIG. 16.—AS A RESULT OF FROST ACTION IN AN OTHERWISE STABLE SHALE STRATUM THOUSANDS OF TONS OF MATERIAL FELL UPON THE HIGHWAY

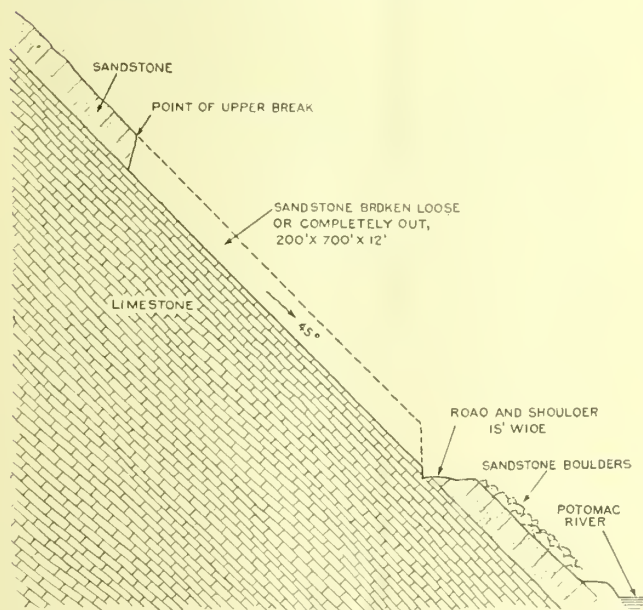


FIG. 17.—DIAGRAM SHOWING HOW BED OF SANDSTONE 12 TO 15 FEET THICK BROKE OUT AND SLID ON A 45° SLOPE AS A RESULT OF MAKING A CUT AS SHOWN

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FIG. 18.—VIEWS LOOKING UP AND ACROSS A 45° SLOPE WHERE A SLIDE OF STRATIFIED ROCK OCCURRED AS A RESULT OF A HIGHWAY CUT

ridge just above the flood plain of the stream in the foreground consists wholly of landslide detritus. A few minor erosion channels have developed in this mass. Material at the toe of this slide has been carried away in times of stream flood, exposing steep sections of detritus not yet covered by vegetation.

A panoramic view is shown in Figure 15, which strikingly illustrates the progressive development of broad concave recessions in Pennsylvanian formations as a result of consecutive landslides over a long period of time, and the consequent development of a unique topography. It is easy to visualize the original skyline of the ridge extending clear across this view. On the left-hand side the ridge has been completely broken through and on the right a great cirque has developed which would already have completely penetrated the ridge had it been narrow at this point.

Water erosion has of course aided this topographic development and has necessarily preceded some of the later lateral landslides. As the original landslide was followed by others so that marked recession of form into the ridge was developed, a drainage area followed. This aided subsequent face-slides and was followed by a series of lateral slides. This sequence accompanied by the removal of material by streams has required only the element of time for so extensive a destruction of the ridge.

Just beyond the building nearest the camera there is a small and very recent slide. A little higher up, outlined in its lower half circumference by a road on one side and path on the other, may be seen the area of a considerable and somewhat older slide. Trees are growing on the site of this slide, which must be from 18 to 25 years old. On the left of the illustration a comparatively recent and extensive slide of a lateral nature is in evidence on the right-hand slope of the hill, once part of the continuous ridge.

SLOPE ADJUSTMENT OF SUPERFICIAL MATERIAL A COMMON TYPE OF SLIDE

Slides of material as a matter of slope adjustment are found over a wide area in western West Virginia, and, particularly among the high hills of both States along the Ohio River, where they often occur in considerable volume. This type of slide is confined largely to the Pennsylvanian formations and consists of a steady dribble of rock fragments and shale. The amount of fall is greatly increased in wet seasons and during alternate freezing and thawing spells. It is not often that the quantity of material at any one spot is particularly large but the accumulation on a stretch of road a few miles long is often several thousand tons. (Figs. 2 and 5.) Most of the material is small and easily handled, but the writer has seen a boulder of massive sandstone weighing many tons bound down a steep slope and, striking the edge of a concrete pavement, dislocate it several inches and split it down the middle for 200 feet or more. No extended discussion as to the underlying causes of this type of slide is needed as it is obvious that they are the same forces and conditions which cause the formation of detrital masses.

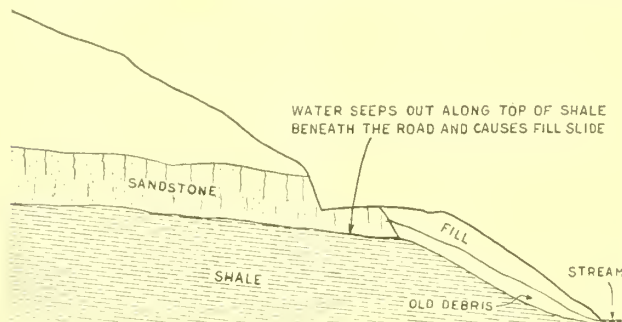


FIG. 19.—ROAD LOCATED SO THAT A SLIDE OF FILL MATERIAL IS ALMOST CERTAIN TO OCCUR. WATER PASSES THROUGH JOINTS IN THE SANDSTONE AND ALONG TOP OF SHALE AND ENTERS FILL BENEATH PAVEMENT

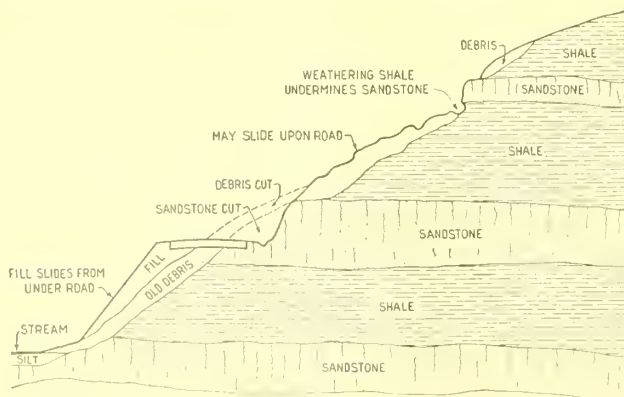


FIG. 20.—CONDITIONS HERE ARE MUCH THE SAME AS IN FIGURE 19 EXCEPT THE ROAD IS PARTLY LOCATED ON SOLID MATERIAL

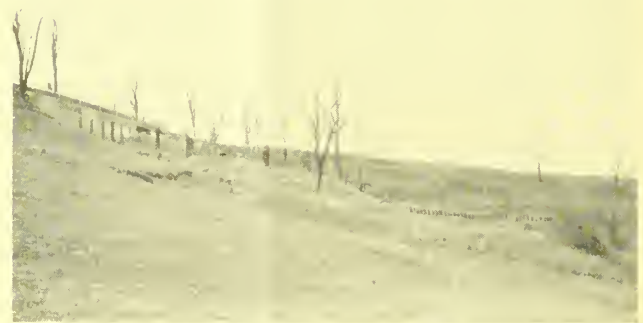


FIG. 21.—TYPICAL FILL SLIDES. IN THE LOWER PICTURE A NATURAL SLIDE IS TO BE SEEN IN THE FIELD BELOW THE FILL SLIDE



FIG. 22.—A FILL SLIDE THAT HAS UNDERMINED A CONCRETE ROAD. OFTEN LARGE SECTIONS OF THE ROAD BREAK OUT AND ARE CARRIED AWAY

SLOPE ADJUSTMENT OF JOINTED MATERIAL CAUSES FEW SLIDES

Slides which are the result of comparatively small-scale adjustments of jointed material where a vertical face results from an artificial cut are found in a few localities in West Virginia. Those observed by the writer were in the eastern part of Mineral County. The most interesting one occurred where, in order to avoid crossing and recrossing a stream which had undercut a massive shale formation until it stood with a vertical face, the stream course had been deflected, a fill made, and a highway constructed at the foot of the shale cliff. The shale mass here, although not a plastic variety, broke off at the top, probably as the

result of freezing of water in joint fractures, and dumped several thousand cubic yards of material on the road. The slope adjustment resulting from this fall is probably sufficient to provide stability of the shale mass above the road for the immediate future. (Fig. 16.)

SLIDES OF ROCK STRATA RARE BUT CAUSE GREAT DAMAGE

The fourth class of slide which might be described as structural slides occurs when stream erosion or an

it at its farthest point. It resulted in the precipitation of an enormous mass of sandstone upon the road and across it. Figure 18 shows two views of this slide.

A slide occurred in Cumberland, in similar formations with like geological structure. This slide was also due to a highway cut and about 5,000 cubic yards of rock was precipitated upon the highway. The disastrous Sand Patch tunnel slide on the Baltimore & Ohio Railroad was of this type.

FILL SLIDES MOST SERIOUS PROBLEM

Probably more than 50 per cent of the damage to roads and the most serious problems in connection with them in these States are caused by fill slides, that is, the subsidence of fill material from under part or all of the road pavement. Fills made of detrital material have the same tendency toward movement as the natural detrital masses and it is possible that this tendency is increased somewhat by being loosened, worked, and placed in the fill with a lesser degree of compaction.

Through fills seldom give trouble, but under exceptional conditions slips occur in them or they "go out" altogether. Sidehill fills go out sometimes as a result of under-pavement seepage, sometimes as a result of defective culverts, and sometimes because of saturation from prolonged rains. Figures 19 and 20 diagrammatically illustrate ways in which under-pavement seepage

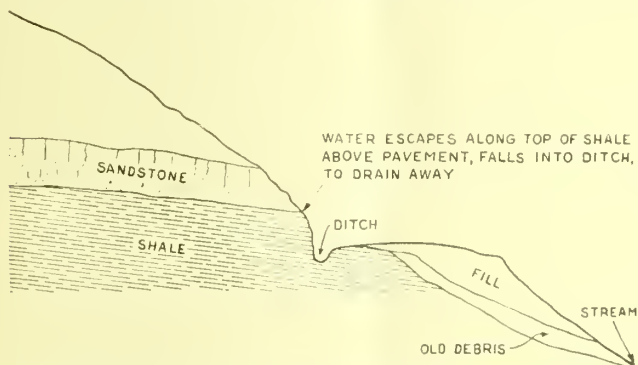


FIG. 23.—ROAD LOCATED SO THAT DITCH INTERCEPTS ALL SEEPAGE WATER UNLESS THE SHALE IS PERMEABLE WHICH IS OF RARE OCCURRENCE. FILL SLIDES SHOULD NOT OCCUR UNDER THESE CONDITIONS

artificial cut removes support from dipping strata, or from rock with dipping cleavage planes. The type is rare in the West Virginia and Ohio region. The best example is found in West Virginia, on the side of a high ridge which overlooks the Potomac River about 1 mile south of Cumberland, Md. At this place beds of massive sandstone lie upon limestone strata, as illustrated in Figure 17, and are probably separated from it by a thin seam of sandy clay. These formations dip toward

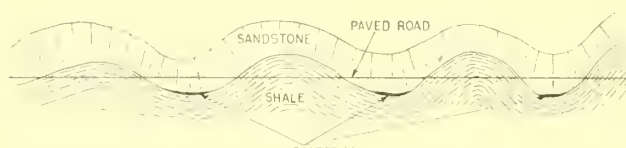


FIG. 25.—SECTION THROUGH FOLDED STRATA OF SANDSTONE AND SHALE WITH SHARPNESS OF FOLDS EXAGGERATED. HORIZONTAL LINE SHOWS POSITION OF HIGHWAY. IN FORMATIONS OF THIS KIND SLIDES HAVE OCCURRED WHERE SEEPAGE ALONG THE UPPER SURFACE OF THE SHALE IS NOT INTERCEPTED BY THE ROAD DITCH

may cause fill slides, and Figures 21 and 22 illustrate slides due to this condition. In these cases the natural seepage channels along the upper surfaces of the shale beds bring about a sufficient degree of fill saturation to cause slides. In the lower illustration of Figure 21 the hill in the background is low, yet it supplied sufficient underground water to cause this fill slide and many natural slides in the field below. Figure 23 shows a condition where seepage water is intercepted by the road ditch and the fill rests on impervious shale. Fill slides should not occur under these conditions.

Figure 24 illustrates a case where the weight of the fill has started a movement of material below and which will soon involve a movement of the fill itself.

A particularly interesting case showing the relation between rock strata and slides was encountered in Mason County, W. Va., and is illustrated in Figure 25. The strata are gently folded in low anticlines and shallow synclines and a road has been located in a sidehill cut in such manner as to bring the impervious shale and the jointed sandstone overlying it alternately above and below the pavement. Theory indicates that in this case seepage water will alternately flow into the highway ditch and into the road fill depending on the position of the undulating surface of impervious shale. Field observations demonstrated this theory to be correct. A series of fill slides was found wherever the shale surface dipped below the ditch and a condition of stability existed where it was above.

(Continued on page 35)



FIG. 24.—UPHEAVAL OF OLD DETRITUS CAUSED BY WEIGHT OF FILL. NEW CRACKS CAN BE SEEN AT BACK OF MAN. THE STABILITY OF THE ROAD UNDER SUCH CONDITIONS IS EXTREMELY UNCERTAIN

the Potomac River at an angle of 45°. A roadway was excavated from solid rock along the side of this ridge. Before the road was completed a large rock slide occurred. Breaks opened at two points not far apart. One of these, several feet in width, extended a long distance diagonally up the hillside, but very little of the sandstone slid far enough to do much damage. The second break extended for a distance of about 700 feet measured along the road, and was about 300 feet above

STANDARD SIZES OF CRUSHED STONE¹

Reported by F. H. JACKSON, Engineer of Tests, Bureau of Public Roads

PRACTICALLY every one having to do with either the production or the use of crushed stone will concede at least the theoretical advantages which may be derived from the standardization of sizes. The wide divergence of existing standards of construction, however, coupled with a natural reluctance on the part of engineers to change their practice simply to comply with a national standard, has made progress in this field very slow. The unsystematic development of the various types of bituminous roads has resulted in a demand for a large number of sizes of stone for a comparatively small number of distinct types of construction, the variations often being of academic rather than practical significance.

LARGE NUMBER OF SIZES OF CRUSHED STONE NOT NEEDED

We may classify these variations in requirements in two groups, (1) those due to distinct differences in engineering practice for a given type of construction, and (2) those very small differences in requirements which are in reality meaningless, but which frequently cause considerable trouble. As an illustration of the first class, a survey of 27 current State specifications for penetration bituminous macadam reveals that there are now specified as many as 10 different sizes of stone for use in the penetration course alone, varying all the way from a 1 to 2 inch size to a 2½ to 3½ inch size. As an illustration of the second class, the requirements for size of chips for bituminous macadam in current specifications show an inexcusable number of slight variations in size, such as $\frac{3}{16}$ to $\frac{5}{16}$ inch, $\frac{1}{4}$ to $\frac{1}{2}$ inch, $\frac{5}{16}$ to $\frac{1}{2}$ inch, and other sizes.

Making every allowance for variations in size necessitated by difference in quality, it is yet obvious that the multiplicity of requirements in force is not only unnecessary and confusing but also works a hardship on the producer, increases the cost of production, and so tends ultimately to increase prices. On the other hand, it must be remembered that it is the engineer in charge of construction and not the producer of the material who is responsible for the quality of the work and he can not be expected to abandon a size with which he has perhaps had many years' satisfactory experience unless he is convinced that the standard size will prove just as satisfactory. And herein lies the greatest difficulty. It is with the engineers and not the producers. Experience with producers in general shows that they are willing to supply what the engineers want, provided the latter will only agree as to just what they do want.

The road materials committee of the American Society for Testing Materials, through its sectional committee on standard sizes of broken stone, broken slag, and gravel, has considered this problem for a number of years and has prepared tentative specifications for commercial sizes of broken stone. These tentative specifications were prepared after an exhaustive study of existing specifications with a view to the selection of the minimum number of primary sizes of crushed stone which would supply the engineers' needs and at the same time eliminate the many small variations in requirements which had no justification

other than that they represented the ideas of individual engineers or groups of engineers. This was no easy task because it involved not only the elimination of many sizes for which the committee felt standard sizes could be substituted, but also because it necessitated a careful study of the various factors which influence the efficiency of plant-screening operations in order that the size limits and tolerances specified might be practical from the standpoint of economical production. The problem required a careful balancing of these practical limitations against theoretical requirements.

SURVEY INDICATES FACTORS AFFECTING SCREENING

Before proceeding to a discussion of the proposed standards, it may be of interest to review briefly the results secured from a rather extensive survey of crushed stone plants made by the Bureau of Public Roads several years ago. The survey included over 100 representative commercial plants situated in the New England, Middle Atlantic, and Ohio Valley States. The information secured, while bearing particularly on the screening operation, included data relative to crushers, their number, type and size, speed and arrangement of conveyers, etc. The screen data included the number, type, and arrangement of screens; the nominal as well as actual size of perforations; the length, diameter, pitch, and speed of revolving screens with the number and length of each section; arrangement of jackets, etc. A record was made of each commercial size produced by the plant with the screen installation as indicated at the time of inspection. This record included the nominal size limits for each product, the name and number under which it was sold, and the specifications it was supposed to meet. Finally, a representative sample of each size was secured from a stock ready for delivery and a screen analysis was made with laboratory screens having circular openings.

From these data it was possible to determine the efficiency of each screening operation at the time of inspection in so far as it was affected by the screens themselves. Other factors which affect screening efficiency, such as fluctuations of the feeding rate and moisture on the stone, were noted and their effect on the particular material selected for sampling determined as nearly as possible. It may be of interest to summarize briefly the conclusions which it was possible to draw from this survey, and they may be stated as follows:

1. The length of a revolving screen influences the grading of the screened product to a marked degree.
2. Within the relatively narrow limits usually found in plant installations pitch and speed of revolving screens apparently have no material influence on grading, probably on account of other predominating factors, such as fluctuation in the rate of feed of stone to the screen, which it is impossible to control in a practical way.
3. The effect of oversize holes due to wear of the screen is practically negligible in view of the relatively large amount of stone held on a revolving screen which theoretically should have passed through it.
4. Small amounts of oversize stone sometimes found in products screened through holes of certain nominal diameter usually are due to faulty bin or chute construction, lack of repair, or other deficiencies in the storing or handling of the material.

¹ Paper read before the tenth annual meeting of the National Crushed Stone Association, Detroit, Mich., Jan. 19, 1927.

5. The grading of the screened product can not be controlled with any degree of certainty by simply specifying the size of openings in the revolving screens over which and through which it shall pass.

6. It is neither practical nor necessary to specify that all material retained on and passing revolving screens of certain sizes shall lie between laboratory screens of the same size.

7. Laboratory screens may be used to control the grading of the plant product if a reasonable tolerance is allowed, that is, one wide enough to cover the recognized inefficiency of the revolving screen and yet close enough to insure sufficiently well-graded materials.

8. Inspection of the results of hundreds of screen analyses indicates that as much as 5 per cent of material should be allowed larger than the size of perforations in the revolving screen through which the product is supposed to pass, and as much as 15 per cent generally should be allowed smaller than the size of the perforations in the revolving screen upon which it is supposed to be retained.

PLANT LIMITATIONS RECOGNIZED

The last conclusion applies, of course, only to products sized with revolving screens and presupposes an adequately designed plant and efficient operation. It applies to the so-called primary sizes only, that is, those sizes the upper and lower limits of which are close together, as $\frac{3}{4}$ to $1\frac{1}{4}$ inches. With combined sizes, such as are used as concrete aggregate, $\frac{1}{4}$ to $2\frac{1}{2}$ inches for example, the tolerance on the lower limit may and should be materially reduced.

With regard to the method of specifying the sizes desired, there are still many engineers who believe that it is necessary to tell the producer just what plant screens to install in order to obtain the sizes they desire. The operator of the average commercial plant is in a much better position to decide on the particular screen installation he needs than the engineer, who should specify only the size or sizes desired in such a manner as to admit of but one interpretation; that is, by reference to laboratory screens. It then becomes the operator's duty to study his installation so as to produce the material in the most efficient manner. Viewed from this angle, the object of the 5 and 15 per cent tolerances just mentioned is simply to recognize the practical limitations beyond which it is impossible to carry efficient operation without greatly increasing the cost.

The facts which were brought out by the survey were of course available for the use of the committee on standard sizes of the American Society for Testing Materials, and form the principal basis of the committee's recommendations as regards tolerances in their proposed specification.

FIVE STANDARD SIZES OF CRUSHED STONE PROPOSED

The committee was, at the outset, confronted with the necessity of reducing the number of primary sizes to a minimum consistent with sound engineering practice. Moreover, it was recognized that the limits of the various primary sizes should not overlap, and when taken together should represent the entire output of a plant. It was also felt that if possible the number of primary separations should be limited to five, due to the greatly increased cost of producing more than five primary sizes in one plant at one time. It is interesting to note in this connection that many large producers replying to a recent invitation from the National Crushed Stone Association to criticize the proposed standard, stated that from the standpoint of economic production, the number of primary sizes should be limited to five.

The committee also considered very carefully the question of nomenclature. What would be the simplest

and most easily understood method of designating sizes? At present there are many systems in use, to the utter confusion of everyone. We have No. 3 stone; we have 1-inch stone, and we have pea stone, to mention three methods of designating sizes and these designations do not always mean the same thing in different localities. No. 1 stone in one State may be described as No. 3 stone in an adjoining State. What does the purchaser mean when he asks for a car of 1-inch stone? Does he mean a maximum size of 1 inch or an average size of 1 inch and if so what are his upper and lower limiting sizes?

The committee after carefully considering the various systems in current use decided that the simplest as well as most definite method of designation would be to specify both the upper and lower limiting sizes, as, for instance, $\frac{1}{4}$ to $\frac{3}{4}$ inch size, $\frac{3}{4}$ to $1\frac{1}{4}$ inch size, etc., which together with a standardized schedule of tolerances and intermediate requirements would give a clear understandable designation, provided a method could be agreed upon for measuring size. There has been a considerable difference of opinion on this point and it has been difficult to bring about agreement between those who favor the square mesh on account of its application in the design of concrete by the fineness modulus method, and those who favor the round hole because it has been used for many years for measuring the size of crushed stone for bituminous road work, and in general is considered a more accurate measure of size than the square opening. However, the committee on road materials of the American Society for Testing Materials, which contains representatives of both sides, last year went on record as favoring the round aperture, and with this precedent the sectional committee on standard sizes decided to submit its tentative schedule of sizes with this method of measuring as the basis. In other words, when we say $\frac{1}{4}$ to $\frac{3}{4}$ inch size, we mean that portion of the product of the crusher at least 85 per cent of which will be retained upon a laboratory screen having circular openings $\frac{1}{4}$ inch in diameter and not more than 5 per cent of which will be retained upon a laboratory screen with circular openings $\frac{3}{4}$ inch in diameter.

On the basis of five primary sizes as the maximum limit and after careful study of existing specifications, the committee proposed the following divisions of the crusher run from 0 to $3\frac{1}{2}$ inches:

0 to $\frac{1}{4}$ inch.
 $\frac{1}{4}$ to $\frac{3}{4}$ inch.
 $\frac{3}{4}$ to $1\frac{1}{4}$ inches.
 $1\frac{1}{4}$ to $2\frac{1}{2}$ inches.
 $2\frac{1}{2}$ to $3\frac{1}{2}$ inches.

These separations are to be on the basis of laboratory screens with round openings. Assuming that, in general, stone will crush in such a way that the percentage of the total crusher run passing any particular size screen will be in proportion to the size of the opening—that is, conforming to a straight-line grading—then the relative percentages of the total crusher run obtained in each of the five primary sizes would be about as follows:

0 to $\frac{1}{4}$ inch—10 per cent.
 $\frac{1}{4}$ to $\frac{3}{4}$ inch—15 per cent.
 $\frac{3}{4}$ to $1\frac{1}{4}$ inches—15 per cent.
 $1\frac{1}{4}$ to $2\frac{1}{2}$ inches—35 per cent.
 $2\frac{1}{2}$ to $3\frac{1}{2}$ inches—25 per cent.

The relative percentages of the various sizes will, of course, vary in individual cases with the kind of stone, type of crushers, and amount of recrushing. For

general conditions, however, and assuming that stone above $3\frac{1}{2}$ inches is rejected and recrushed, the above may be considered an indication of average results. A recent report,² based on elaborate plant tests, substantially confirms the above figures.

PROPOSED SIZES MEET ALL ORDINARY REQUIREMENTS

The above system of sizes was considered with a view to ascertaining to what extent each size would be available to the engineer and how the sizes could be combined, remembering that for each primary size a tolerance of 15 per cent is allowed on the lower limit and 5 per cent on the upper limit. The problem of establishing standard size limits so as to care for the entire output of all plants with a minimum of waste at all times is, of course, a hopeless one. There are uncontrollable fluctuations in demand which may cause a certain size to be in great demand one month and a drug on the market the next. There may be a difference in the demand for stone of different kinds. The supply of limestone in certain sizes required for road work or as concrete aggregate may be greatly affected by the demand for flux stone or agricultural limestone, while trap rock would not be affected by the latter demands. It is obviously impossible to carry standardization to the point where a uniform standard of plant practice is possible. Every plant and every producing district has its own peculiar problems and the best that standardization can do is to provide a common measure for the use of the engineer in making known his needs to the producer so that the latter will know exactly what the former wants and the former will know exactly what the latter has to offer.

Considering the uses to which the several suggested sizes may be put, let us take first the $\frac{1}{4}$ to $\frac{3}{4}$ inch size. This is commercial $\frac{1}{2}$ -inch stone, very largely used in road work as chips in bituminous macadam construction by the penetration method, in certain grades of bituminous concrete, as a surface dressing, and in maintenance work. It was felt that a tolerance of 15 per cent on the lower limit gave a reasonably well sized product, and was liberal enough for economic production. The next size, $\frac{3}{4}$ to $1\frac{1}{4}$ inch, is commercial 1-inch stone, with the same tolerance of 15 per cent. It may be used alone as the intermediate course in bituminous macadam, or in combination with the $\frac{1}{4}$ to $\frac{3}{4}$ inch size in certain grades of bituminous concrete or as an aggregate in Portland cement concrete, where the maximum size must not exceed $1\frac{1}{4}$ inches. A combination size made up of the $\frac{1}{4}$ to $\frac{3}{4}$ inch size and the $\frac{3}{4}$ to $1\frac{1}{4}$ inch size, each of which complies with specifications for that size, may reasonably carry a somewhat lower tolerance than 15 per cent—10 per cent at most and possibly 5 per cent in the case of concrete aggregate where a relatively large percentage of the $\frac{3}{4}$ to $1\frac{1}{4}$ inch size should be used in the mixture.

The next size, $1\frac{1}{4}$ to $2\frac{1}{2}$ inches, is proposed for use in macadam road construction, either penetration or waterbound. Objection has been raised to this size for penetration macadam on the ground that a larger size, say, 2 to 3 inches, would give better results and that the maximum size stone should be limited only by the depth of the course. Reviewing the several State highway department specifications, we find a wide divergence of practice. Out of 27 State specifica-

tion standards, there are 3 with a maximum size of 2 inches, 12 with a maximum of $2\frac{1}{2}$ inches, 7 with a maximum of $2\frac{3}{4}$ inches, 5 with a maximum of 3 inches, and 1 with a maximum of $3\frac{1}{2}$ inches, the latter to be used only in courses exceeding 3 inches in depth. This indicates the difficulties in the way of pleasing everybody. It may be suggested that a compromise could be made by raising the upper limit to 3 inches without realizing that to do so would throw the entire specification out of balance by eliminating the $2\frac{1}{2}$ -inch separation which is insisted upon by the users of concrete aggregates. At the present time the committee is inclined to adhere to the present limit of $2\frac{1}{2}$ inches for bituminous macadam where the course is less than 3 inches in depth and to suggest the $2\frac{1}{2}$ to $3\frac{1}{2}$ inch size where the course is greater than 3 inches in depth.

STANDARD SIZES SUITABLE FOR CONCRETE AGGREGATE

We now have three primary sizes which together represent the run of the crusher from $\frac{1}{4}$ inch to $2\frac{1}{2}$ inches. When properly combined, these three sizes may be used to produce an ideally graded aggregate for concrete for pavements where the maximum size is $2\frac{1}{2}$ inches. Just what constitutes an ideal grading for coarse aggregate for pavements makes an interesting problem for discussion, both from an economic and an engineering standpoint. Recent investigations made by the Bureau of Public Roads in cooperation with the New Jersey State Highway Commission indicate that the yield of concrete from given volumes of the constituent materials is greatly influenced by the grading of the coarse aggregate. As an illustration, it was found that in the case of a $1:1\frac{3}{4}:3\frac{1}{2}$ mix, using crushed trap rock as coarse aggregate, a stone graded uniformly from $2\frac{1}{2}$ inches to $\frac{1}{4}$ inch in size required 6.18 bags of cement per cubic yard of finished concrete as compared to 6.72 bags required when the material under 1 inch in size was omitted—a saving in cement of about one-half bag per cubic yard. It is true that a considerably greater weight of stone was required in the case of the well-graded aggregate than in the case of the poorly graded material, so that from a strictly economic point of view the gain in cement would be somewhat offset by the additional stone required. The economic aspects of concrete proportioning have not received the attention that they should.

If we assume for the moment that the ideal grading curve for stone between $\frac{1}{4}$ inch and $2\frac{1}{2}$ inches in size is approximately a straight line, which according to Taylor and Thompson is the condition of maximum density, it would be necessary to mix the three commercial sizes in approximately the following proportions:

$1\frac{1}{4}$ to $2\frac{1}{2}$ inch size—60 per cent.
 $\frac{3}{4}$ to $1\frac{1}{4}$ inch size—25 per cent.
 $\frac{1}{4}$ to $\frac{3}{4}$ inch size—15 per cent.

This is very nearly the ratio in which these sizes are produced in the normal crushing process, and would therefore be the most economical way for the producer to ship, provided he did not have a heavy demand for $\frac{1}{4}$ to $\frac{3}{4}$ inch size, which is often the case. However, the point to be emphasized here is that in concrete work it is highly important to have a uniform grading of coarse aggregate from day to day if satisfactory results are to be obtained. This rule holds irrespective of the method used in designing the mix. As a matter of fact, it is even more important viewed in the light of some of the new theories of concrete design which have been advanced, due principally to the effect of

² A Study of the Mechanical Analysis of Crushed Stone, the Crushed Stone Journal, July 1, 1926, published by the National Crushed Stone Association, Washington, D. C.

variations in gradation on the workability of the concrete and consequently on the amount of water which must be used in order to properly place the mix. Variation in the amount of water used in the mix will affect the strength of the concrete, resulting in a non-uniform product. From the viewpoint of the producer it is of great importance to maintain uniformity in the gradation of aggregates for concrete even though the specification may allow a considerable variation in some of the intermediate sizes. Rigid attention to such details will do more to overcome the natural disadvantages of crushed rock as compared to certain other aggregates than will any other one thing and will at the same time insure better concrete.

The remaining size in the crusher-run product is from $2\frac{1}{2}$ to $3\frac{1}{2}$ inches, and would be available for use either as base course in waterbound or as bituminous macadam for courses over 3 inches in depth. In times of heavy demand for the smaller sizes, this size could, of course, be recrushed as desired.

PROBLEM NOT YET ENTIRELY SOLVED

The original tentative standard specifications of the American Society for Testing Materials have been amended once by the insertion of a $0\frac{1}{2}$ -inch size to provide stone for Topeka-type pavement. Many additional changes have been suggested both voluntarily and in response to inquiries sent out by the committee. Among them is a request for a commercial $\frac{3}{4}$ -inch stone, from $\frac{1}{2}$ inch to 1 inch in size, for use in tar macadam. The sponsors for this size claim that the $\frac{3}{4}$ to $1\frac{1}{4}$ inch size is too large for intermediate-size stone in bituminous macadam. Here again engineering opinion causes difficulty in the adoption of a rational standard, because it can readily be seen that the $\frac{1}{2}$ to 1 inch size has no place in the system as above outlined.

Strong representation has also been made by certain eastern groups that a $\frac{5}{8}$ -inch limiting size be substituted for the $\frac{3}{4}$ -inch limit. This size is very largely used in the East now as the lower limiting nominal size for cement concrete aggregate, and there seems some merit to the contention that with the $\frac{1}{4}$ -inch lower limit there is danger of obtaining too high a percentage of small stone which makes a very harsh concrete, difficult to finish properly. The above illustrations are given simply to show that the problem has by no means been solved and will require a great deal more work on the part of intelligent, sympathetic representatives of both the producing and the consuming interests before it is solved.

It should not, however, be assumed that no progress has been made since this matter first came up for discussion. The sizes as proposed have been adopted as tentative by the American Association of State Highway Officials, the Federal Specification Board and the Asphalt Association. It will be seen, therefore, that the specifications have advanced considerably beyond the initial stage. The specifications as adopted by these agencies, however, with the exception of those of the

Federal board, are in the nature of typical rather than governing specifications, and are in no sense mandatory.

In conclusion, there seem to be three possible courses which may be followed toward the ultimate solution of this vexing question. We may work for the standardization of the suggested series of sizes; or failing that, suggest other sizes which may be substituted therefor; or failing that, abandon all idea of adopting a national standard, acknowledging that the question of the actual sizes to be used is a problem for each producing district to work out in conjunction with the consumers it serves. In the latter event, the committee of the American Society for Testing Materials would confine its attention to the standardization of methods of designation, standard tolerances, and standard methods of measuring size.

CORRECTION

In the March issue of Public Roads in the article "The trend of highway design" a radical sign was omitted in printing the formula on page 8 for road widening on curves. This formula should have been printed as follows:

$$W = 2(R - \sqrt{R^2 - L^2}) + \frac{35}{\sqrt{R}}$$

In which W = Widening in feet.

R = Radius of curve in feet.

L = Wheel base of vehicle in feet.

(Twenty feet recommended.)

In the January issue, in the discussion of earth pressures on culvert pipes by Dean Anson Marston, the following legend should be substituted for the one printed under Figure 2c on page 227:

Yielding of foundation and shortening of vertical diameter is such that the settlement of the fill material directly over the conduit is greater than in the fill alongside the conduit only above a level somewhat below the top of embankment. Below that level the settlement alongside is equal to or greater than the settlement directly above the conduit.

(Continued from page 31)

The various types of landslides which have been described are found in a large area embracing many hundreds of miles of highway. Past experience has been that they not only require the constant expenditure of large sums of money which might go into the improvement of other roads, but what will be of ever-increasing importance, they are serious obstacles to the flow of highway traffic. If ordinary road construction and slide control methods are followed, a continuation of landslide troubles may be expected indefinitely, especially of those occurring under half-fill roads. As a result of the studies which have been made it is believed that much can be done to improve these conditions. The problem is of sufficient importance to deserve exhaustive detailed study, even though it results only in partially correcting or alleviating troubles resulting from landslides.

MOTOR VEHICLE REGISTRATION AND REVENUE, 1926

Reported by L. A. ABBOT, Associate Statistical Engineer, United States Bureau of Public Roads

During the calendar year 1926 there were over 22,000,000 motor vehicles registered in the United States or more than double the registration of 1921 and four times that of 1917. The year's registration represents an increase of 10.3 per cent or slightly over 2,000,000 more than that of 1925. Of the total number of vehicles registered 19,237,171 were passenger automobiles, taxis, and busses, and 2,764,222 were motor

bond issues. The remainder was used for payment of collection costs and miscellaneous purposes.

Figure 1 shows graphically the number of motor vehicles registered in the United States from 1910 to the present time. In 1895 four automobiles were produced and the number increased steadily to 1910 when approximately 460,000 motor vehicles were registered. From 1910 the curve rises rapidly upward to 1915 and then even more rapidly to 1923. Since 1923 the curve has declined slightly in steepness but it is still much steeper than at any period prior to 1923. The curve does not show any considerable flattening such as would indicate the near approach to a saturation point in the United States as a whole.

Curves representing the registration of passenger vehicles and trucks separately are also shown. The percentage of trucks to total vehicles in 1926 was 12.5 per cent as compared with 9.4 per cent in 1921. The greatest increase in truck registration was in 1924 when there was an increase of 580,459 or 37.3 per cent over 1923. The 1925 and 1926 increases have been 308,681 and 322,513, respectively, which indicates continued growth of commodity transportation by highway.

Table 1 shows the increase in registration each year since 1921, the accumulated increase since 1921 and these figures expressed as a percentage of the 1921 registration. The last column shows the year 1923 to be the one of largest increase with 27.2 per cent and successively smaller percentages in each of the years following. Figure 2 shows the same facts graphically with 1920 as a base.

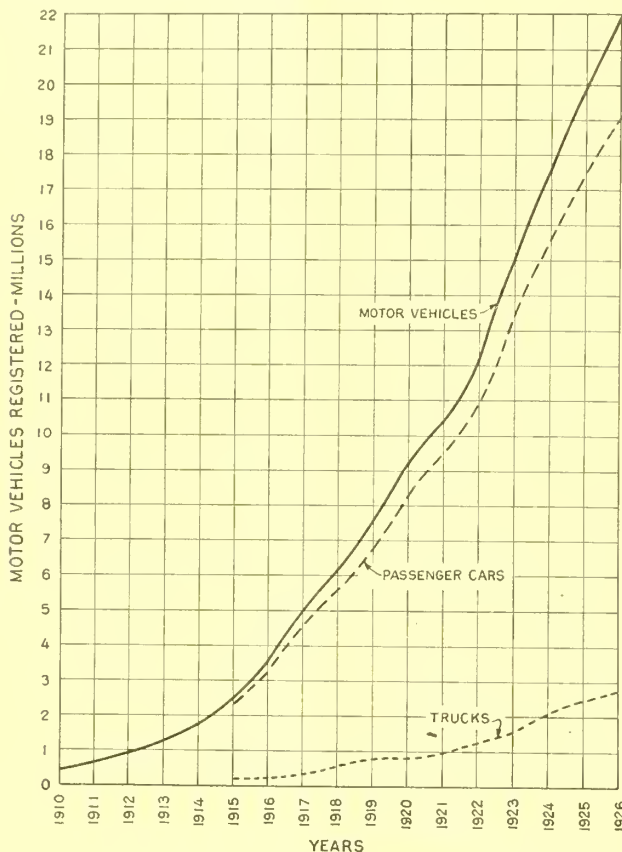


FIG. 1.—MOTOR VEHICLE REGISTRATIONS 1910–1926

trucks and road tractors. Complete details of motor vehicle registration and taxation as collected by the bureau are given in Tables 9 and 10.

Florida, with an increase of 40.2 per cent, excluding nonresident registrations as in other States, showed a greater gain than any other State. Oklahoma, with a gain of 17.8 per cent, and second only to Florida in respect to the percentage of increase, was followed closely by Alabama, Idaho, Louisiana, Mississippi, and Utah, all of which had increases over 15 per cent.

The receipts from registration fees, licenses, and so forth, amounted to \$288,282,352 as compared with \$260,619,621 in 1925. Of the gross receipts \$191,111,302 was available for highway purposes under the supervision of the State highway departments, \$51,702,184 was allocated to counties for expenditure on local roads, and \$25,274,178 was used for payments on highway

TABLE 1.—Motor vehicles registered each year since 1921, accumulated increase since 1921 and annual increase compared on a percentage basis with the 1921 registration

Year	Total motor vehicles registered	Increase over previous year	Accumulated increase over 1921	Accumulated increase as a percentage of 1921 registration	Annual increase as a percentage of 1921 registration
				Per cent	Per cent
1921.....	10,463,295				
1922.....	12,238,375	1,775,080	1,775,080	17.0	17.0
1923.....	15,090,936	2,852,561	4,627,641	44.2	27.2
1924.....	17,593,677	2,502,741	7,130,382	68.1	23.9
1925.....	19,937,274	2,343,597	9,473,979	90.3	22.2
1926.....	22,001,393	2,064,119	11,538,098	110.2	19.9

TABLE 2.—Accumulated increases in motor-vehicle registrations since 1921 expressed as a percentage of the 1921 registration by geographic divisions

Geographic division	1922	1923	1924	1925	1926
	Per cent	Per cent	Per cent	Per cent	Per cent
New England.....	11.7	35.7	62.0	82.7	95.7
Middle Atlantic.....	22.5	51.0	77.3	99.2	120.8
East North Central.....	18.0	46.4	69.3	89.0	107.7
West North Central.....	11.2	28.1	41.0	57.2	69.3
South Atlantic.....	17.9	50.1	82.5	113.5	145.0
East South Central.....	16.9	53.9	85.7	124.3	153.7
West South Central.....	15.4	49.2	78.7	114.5	139.4
Mountain.....	8.8	30.6	48.2	69.2	84.0
Pacific.....	22.6	55.0	83.7	102.0	123.2
United States.....	17.0	44.2	68.1	90.3	110.2

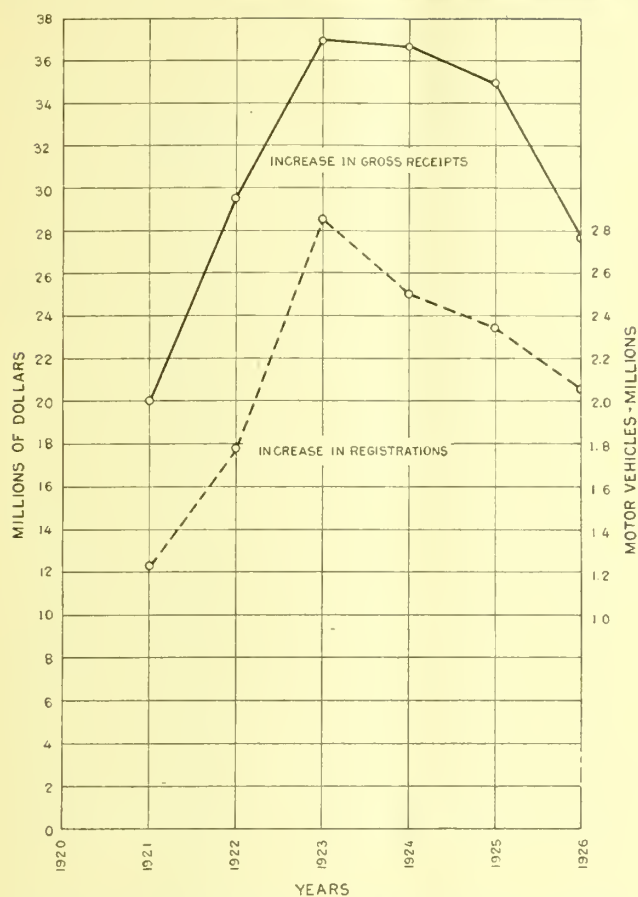


FIG. 2.—ANNUAL INCREASES IN MOTOR VEHICLE REGISTRATIONS AND REGISTRATION REVENUE OVER PRECEDING YEAR

TABLE 3.—Annual increases in motor-vehicle registrations expressed as a percentage of the 1921 registration by geographic divisions

Geographic division	1922	1923	1924	1925	1926
	<i>Per cent</i>	<i>Per cent</i>	<i>Per cent</i>	<i>Per cent</i>	<i>Per cent</i>
New England.....	11.7	24.0	26.3	20.7	13.0
Middle Atlantic.....	22.5	28.5	26.3	21.9	21.6
East North Central.....	18.0	28.4	22.9	19.7	18.7
West North Central.....	11.2	16.9	12.9	16.2	12.1
South Atlantic.....	17.9	32.2	32.4	31.0	31.5
East South Central.....	16.9	37.0	31.8	38.6	29.4
West South Central.....	15.4	33.8	29.5	35.8	24.9
Mountain.....	8.8	21.8	17.6	21.0	14.8
Pacific.....	22.6	32.4	28.7	18.3	21.2
United States.....	17.0	27.2	23.9	22.2	19.9

TABLE 4.—Comparison of per capita wealth and number of persons per motor vehicle arranged in order of the latter by geographic divisions¹

Geographic division	Property wealth per capita, 1922	Motor vehicles per 1,000 of population		Per cent increase in motor vehicles from 1921 to 1926	Persons per motor vehicle	
		1921	1926		1921	1926
Pacific.....	\$3,934	170	226	123	5.9	3.1
West North Central.....	3,588	147	241	69	6.8	4.1
East North Central.....	3,063	118	223	108	8.5	4.5
Mountain.....	3,435	116	187	84	8.6	5.4
West South Central.....	1,857	80	174	139	12.5	5.7
New England.....	3,186	94	170	96	10.7	5.9
Middle Atlantic.....	3,352	78	159	121	12.8	6.3
South Atlantic.....	2,005	63	141	145	15.8	7.1
East South Central.....	1,437	44	106	154	22.9	9.4
United States.....	2,918	97	188	110	10.3	5.3

¹ Bureau of the Census estimate of population for 1921 and 1926 used in compiling table.

TABLE 5.—New motor vehicles registered¹

Year	New production of motor vehicles	Motor vehicles exported	Production remaining in United States	Motor vehicles imported	Net new motor vehicles to be registered
1921.....	1,724,241	38,430	1,685,811	522	1,686,333
1922.....	2,254,088	78,234	2,175,854	483	2,176,337
1923.....	3,673,087	151,894	3,521,193	853	3,522,046
1924.....	3,763,649	178,732	3,584,917	604	3,585,521
1925.....	3,768,410	302,931	3,465,479	678	3,466,157
1926.....	4,621,774	305,256	4,316,518	813	4,317,331

¹ Data from United States Department of Commerce. New production figures are for years ending Sept. 30, as cars manufactured after this date do not ordinarily appear in registration figures until the following year. Import and export figures are for calendar year.

TABLE 6.—New motor vehicles to be registered, discarded motor vehicles replaced and relation to new production and total registered

Year	Net new motor vehicles to be registered	Increase in motor vehicle registration	Motor vehicles discarded	Discarded motor vehicles as a percentage of	
				New vehicles registered	Total registered
				<i>Per cent</i>	<i>Per cent</i>
1921.....	1,686,333	1,231,354	454,979	27.0	4.3
1922.....	2,176,337	1,775,080	401,257	18.4	3.2
1923.....	3,522,046	2,852,561	669,485	18.7	4.4
1924.....	3,585,521	2,502,741	1,082,780	30.1	6.1
1925.....	3,466,157	2,343,597	1,122,560	35.2	5.6
1926.....	4,317,331	2,064,119	2,253,212	52.2	10.2
Total or average.....	18,753,725	12,769,452	5,984,273	31.9	---

TABLE 7.—Motor vehicle revenue 1920-1926

Year	Gross receipts	Increase over previous year		Revenue compared with that of 1920	Year's increase	Receipts per motor vehicle	
		Amount	Per cent			Average	Percentage of average amount in 1920
				<i>Per cent</i>	<i>Per cent</i>		<i>Per cent</i>
1920.....	\$102,546,212			100		\$11.10	100
1921.....	122,478,654	\$19,932,442	19.4	120	20	11.70	105
1922.....	152,047,824	29,569,170	24.1	148	28	12.32	111
1923.....	188,970,992	36,923,168	24.3	184	36	12.52	113
1924.....	225,630,760	36,659,768	19.4	220	36	12.82	115
1925.....	260,619,621	34,988,861	15.5	254	34	13.07	118
1926.....	288,282,352	27,662,731	10.6	281	27	13.10	118

TABLE 8.—Disposition of motor vehicle gross receipts for the years 1925 and 1926

Purpose	1925 ¹		1926	
	Amount	Per cent	Amount	Per cent
State highways.....	\$177,666,595	68.1	\$191,111,302	66.2
Local roads.....	46,041,624	17.7	51,702,184	17.9
State road bonds.....	11,819,689	4.5	19,861,995	6.9
County road bonds.....	7,304,325	2.8	5,412,163	1.9
Miscellaneous.....	3,563,087	1.4	3,592,497	1.3
Administration and collection.....	14,224,301	5.5	16,602,211	5.8
Total.....	260,619,621	100.0	288,282,352	100.0

¹ Slight revision has been made in the 1925 figures as first published on account of later and more complete information.

TABLE 9.—Motor vehicle registrations for the year 1926

State	Registered vividually owned ¹	motor vehicles and commercially	Other registered vehicles		Tax-exempt official vehicles and motor cycles			Number of licenses or permits (autos)			Total registered motor cars and trucks, 1925	Year's increase in motor vehicle registrations			
	Total registered motor cars and trucks	Passenger auto- mobiles, taxis, and busses	Motor trucks and road tractors	Trail- ers ²	Motor cycles	United States cars	State and local cars	Motor cycles	Dealers	Operators		Chauffeurs	Number	Per cent	
Alabama	225,930	197,983	27,947	983	401	167			447		1,813	194,580	31,350	16.1	
Arizona	73,682	63,294	10,388		337	176	675		232	86,221	192	68,029	5,653	8.3	
Arkansas	209,419	179,480	29,939	1,584	279	39	629		527		5,131	183,589	25,830	14.1	
California	1,600,475	1,384,152	216,323	30,818	10,363	1,217	20,248	415	3,194	112,170	(6)	1,440,541	159,934	11.1	
Colorado	248,613	227,708	20,905	786	1,480	283	(5)		3,400	21,967	7,762	240,097	8,516	3.5	
Connecticut	263,235	222,283	40,952	326	3,108	71	2,326		5,231	292,253		250,669	12,566	5.0	
Delaware	44,834	36,246	8,588	199	342	44			630	45,067	4,088	40,140	4,694	11.7	
Florida	401,562	331,892	69,670	10,000	1,390	75	2,926	180	3,295		10,854	286,388	115,174	40.2	
Georgia	277,468	241,949	35,519		841	934			864		3,197	248,093	29,375	12.3	
Idaho	44,760	86,339	8,421	180	483	103	1,125	26	402		414	81,506	13,254	16.3	
Illinois	1,370,303	1,195,897	174,606	3,258	6,156	979	(11)	(12)	4,703			93,368	1,263,177	107,326	8.5
Indiana	772,326	665,126	107,200	5,697	3,738	3,184	3,593		2,489			39,828	725,410	46,916	6.5
Iowa	698,998	648,218	50,780	153	1,934	44	2,700	79	2,291			11,047	659,202	39,796	6.0
Kansas	491,276	441,373	49,903	1,221	1,330	192	2,150	50	2,534			457,033	34,243	7.5	
Kentucky	281,557	252,632	28,925	(13)	672	90	1,444	40	1,162		9,299	261,647	19,910	7.0	
Louisiana	239,500	204,000	35,500		500	209						207,000	32,500	15.7	
Maine	151,486	124,158	27,328	871	1,124	64	1,050	71	1,210	173,917	6,327	140,499	10,987	7.8	
Maryland	252,852	240,743	12,109	634	4,039	1,969			6,027	37,938	39,816	234,247	18,605	7.9	
Massachusetts	690,190	593,234	96,956	464	9,215	556	900		2,134	763,951		646,153	44,037	6.8	
Michigan	1,118,785	969,686	149,099	13,628	3,438	371	5,875	(12)	2,133	224,697	81,382	989,010	129,775	13.1	
Minnesota	630,285	559,128	71,157	2,666	2,551	252	306		2,193			569,694	60,591	10.6	
Mississippi	205,200	184,133	21,067	757	92	74	(11)	(12)	5,468			177,262	27,938	16.3	
Missouri	654,554	587,856	66,698	1,489	2,005	311	1,350	5	2,293	5,242	24,933	604,166	50,388	8.3	
Montana	103,958	88,840	15,118		192	229	1,026	11	447		384	94,656	9,302	9.8	
Nebraska	366,773	337,989	28,784	15	1,268	226			2,834			338,719	28,054	8.3	
Nevada	24,014	19,300	4,714	60	82	42	379		110			21,169	2,845	13.4	
New Hampshire	89,001	78,400	10,601	535	1,444	22	300		484	66,600	32,550	81,498	7,503	9.2	
New Jersey	651,415	531,702	119,713	1,602	7,235	708	5,251		2,460	745,659		580,554	70,861	12.2	
New Mexico	54,996	53,173	1,823	143	200	156	527	5	145			49,111	5,885	12.0	
New York	1,815,434	1,508,314	307,120	6,175	18,303	1,666	10,847		4,414	1,568,540	554,769	1,625,583	189,851	11.7	
North Carolina	163,855,047	352,217	32,830	500	870	429	4,110		8,157			340,287	44,760	13.1	
North Dakota	157,822	145,571	12,251		305	3	122					144,972	12,850	8.9	
Ohio	1,480,246	1,295,020	185,226	17,915	12,130	2,362	7,418	278	3,777		4,703	1,346,400	133,846	9.9	
Oklahoma	499,938	449,955	49,983		719	530			604			424,345	75,593	17.8	
Oregon	233,568	214,946	18,622	812	2,123	141	1,754	92		44,677	15,328	216,553	17,015	7.9	
Pennsylvania	1,455,184	1,264,453	190,731	3,313	13,672	1,383	3,047	937	28,167	1,637,188		1,330,433	124,751	9.4	
Rhode Island	110,746	91,798	18,948	45	1,303	56	577	50	278	126,630		101,756	8,990	8.8	
South Carolina	181,189	163,551	17,638	1,006	270	91	5,048		519	152		168,496	12,693	7.5	
South Dakota	168,230	153,840	14,390	(13)	751	132	2,066		1,051			168,028	202	0.1	
Tennessee	279,639	254,342	25,297		751	132	2,066		599			244,626	35,013	14.3	
Texas	1,049,869	944,905	104,964	6,920	2,679	2,505			3,635		10,978	975,083	74,786	7.7	
Utah	85,380	72,880	12,500	430	576	173	705					73,427	11,953	16.3	
Vermont	74,063	68,524	5,539	133	606	28			700	66,154	13,432	69,576	4,487	6.4	
Virginia	322,614	273,764	48,850	437	2,125	1,141	2,551	471	3,915		8,700	282,650	39,964	14.1	
Washington	363,279	310,386	52,893	1,826	2,740	637	4,260	153	4,914	427,507		328,442	34,837	10.6	
West Virginia	227,836	201,645	26,191	329	1,273	33	2,007		12,011	60,355	27,000	217,589	10,247	4.7	
Wisconsin	662,282	581,994	80,288	(13)	3,107	92	484	76	2,785	681,860		594,386	67,896	11.4	
Wyoming	49,883	44,358	5,525		179	209	212		293			47,711	2,172	4.6	
District of Columbia	111,497	97,794	13,703		1,327	837	2,006	194	1,906	70,146		103,092	8,405	8.2	
Totals	22,001,393	19,237,171	2,764,222	99,430	131,546	33,179	102,762	3,133	137,064	7,258,831	1,007,295	19,937,274	2,064,119	10.3	

¹ The first three columns record the regularly registered motor cars and trucks which pay the regular license fees eliminating reregistrations and nonresident owners' cars. The grand total of the first column is subdivided as indicated, passenger cars being shown in second column and freight service trucks and road tractors in the third column. Some States, as noted, classify busses with trucks. Special tables showing the extent and kinds of bus service from non-Government sources can be found in the February, 1927, issue of "Bus Transportation."

² Some States include trailers with motor trucks, as noted.

³ Busses included with trucks.

⁴ Excludes 2,232 tax-exempt trailers, of which 1,499 are owned by public-service corporations.

⁵ Includes 8,278 cars and trucks of public-service corporations exempt by law.

⁶ Included with operators.

⁷ Includes road tractors.

⁸ Included with registered cars, as all pay regular license.

⁹ Nonresident registrations estimated and excluded from figures reported.

¹⁰ Estimated and excluded from motor trucks as reported.

¹¹ Included with private cars at a special low fee.

¹² Included with private motor cycles.

¹³ Not registered.

¹⁴ As reported on July 1, 1926.

¹⁵ Includes only trucks and busses over 3,000 pounds in weight; others classed as passenger cars.

¹⁶ Figures estimated for last six months of calendar year, as registration begins on July 1.

¹⁷ Estimated as trailers and motor cycles reported together.

¹⁸ Excludes 16,274 agricultural tractors reported.

¹⁹ Corrected figures as previously published total included reregistrations in Utah.

²⁰ As reported by Bureau of Budget, and includes 7,859 cars-at-large, not allocated to any State.

Tables 2 and 3, respectively, show the accumulated increases in registration since 1921 and the annual increases expressed as a percentage of the 1921 registration for the various geographic divisions.¹ The East South Central group shows the greatest total increase, the West North Central group the least. During the year 1926 the greatest increases in proportion to the 1921 registration were in the South

¹ The States included in the several geographic divisions are:
New England—Maine, New Hampshire, Vermont, Massachusetts, Rhode Island, Connecticut.

Middle Atlantic—New York, New Jersey, Pennsylvania.

East North Central—Ohio, Indiana, Illinois, Michigan, Wisconsin.

West North Central—Minnesota, Iowa, Missouri, North Dakota, South Dakota, Nebraska, Kansas.

South Atlantic—Delaware, Maryland, Virginia, West Virginia, North Carolina, South Carolina, Georgia, Florida, District of Columbia.

East South Central—Kentucky, Tennessee, Alabama, Mississippi.

West South Central—Arkansas, Louisiana, Oklahoma, Texas.

Mountain—Montana, Idaho, Wyoming, Colorado, New Mexico, Arizona, Utah, Nevada.

Pacific—Washington, Oregon, California.

Atlantic and East South Central groups and the least in the West North Central, New England, and Mountain groups.

Table 4 shows the per capita wealth in the various geographic groups in 1922, and the number of motor vehicles per 1,000 of population, and numbers of persons per motor vehicle in 1921 and 1926. The groups are arranged in the order of the number of motor vehicles per 1,000 of population in 1926 and there is a very evident but not exact relation to the per capita wealth. The table shows the groups to be in the same relative order with regard to number of persons per motor vehicle that they were in 1921, except that the West South Central group has passed the New England group. The Pacific group is conspicuous as having the highest number of motor vehicles per 1,000 of population and the greatest per capita wealth.

TABLE 10. —Receipts from motor vehicle registration fees, etc., for the year 1926

State	Total gross receipts	Subdivision of registration receipts ¹					Miscellaneous receipts					Disposition of gross receipts			
		Motor car receipts			Other vehicles		Dealers' license	Chauffeur and operator permits	Miscellaneous	Collection and administration	For rural highway purposes			For other purposes	
		Total from motor cars	Passenger cars and busses	Trucks and tractors	Trailers	Motor cycles					State highways	Local roads	State and county road bonds ²		
Alabama	\$2,889,252	\$2,873,119					\$3,304	\$9,243	\$3,586	\$111,262	\$550,760	\$560,258	\$1,666,835		\$137
Arizona	467,795	422,490				\$596	4,543	4,294	35,872	18,500	449,295				
Arkansas	3,656,368	3,611,161					5,798	23,911	15,498	73,433	1,176,405	604,367	1,802,163		
California	8,477,233	7,201,314	\$4,528,157	\$2,673,157	\$245,420	37,167	45,120	285,549	662,663	1,563,533	3,721,415	3,192,285			
Colorado	1,507,379	1,455,452	1,228,841	226,611	1,209	2,588	26,738	21,108	284	75,369	716,005				
Connecticut	6,220,668	4,751,388	3,675,495	1,075,893	7,620	13,485	70,465	884,894	492,816		6,220,668				
Delaware	775,577	592,964	425,207	167,757	3,167	1,324	7,105	147,465	23,552		775,577				
Florida	6,764,468	6,702,911				5,302	33,351	16,281	6,623	122,111	4,957,406	1,684,951			
Georgia	3,381,725	3,323,296	2,769,453	553,843		3,888	41,970	6,140	6,431	119,703	3,262,022				
Idaho	1,385,930	1,343,991	1,127,940	216,051	4,190	2,415	23,895	828	10,611		143,178	1,242,752			
Illinois	14,047,208	13,108,642	9,996,135	3,112,507	57,936	20,800	91,072	370,314	398,441		8,536,365		5,510,843		
Indiana	5,093,176	4,744,318	3,683,211	1,061,107	27,233	6,899	48,100	37,395	229,231	270,307	4,822,869				
Iowa	10,208,416									357,294	9,851,122				
Kansas	4,803,130									247,584	2,666,828	1,888,718			
Kentucky	4,131,745	4,007,551	3,142,977	864,574		5,103	32,630	17,761	68,700	174,358	3,525,100	432,287			
Louisiana	3,993,466	3,836,324							157,142	93,508	3,845,533		54,425		
Maine	2,355,365	1,804,455	1,425,202	379,253	2,859	6,540	44,220	377,854	119,437	8,300,899	1,400,893		653,663		527,088
Maryland	2,928,268	2,270,658	1,993,281	277,377	15,057	13,963	29,209	195,324	404,057	292,827	2,108,353				
Massachusetts	13,077,857	10,586,446	7,134,754	3,451,692	20,011	41,675	109,666	1,527,902	792,157	1,028,534	12,049,023				
Michigan	16,953,685	15,468,556	11,647,731	3,820,825	169,568	13,274	94,462	265,733	942,092	1,207,825	9,745,860	6,000,000			
Minnesota	9,976,560	9,855,487	8,076,504	1,778,983	13,026	10,010	39,338	20,736	37,963		6,366,560		3,510,000		
Mississippi	1,973,412	1,966,379	1,787,310	179,069	6,435	598				59,202		1,914,210			
Missouri	7,903,025									1,391,041	2,860,049		3,651,935		
Montana	1,029,383	895,605	751,875	143,730		1,178	33,825	626	98,149	39,788		989,595			
Nebraska	3,636,097	3,463,691	2,956,116	507,575		4,103	40,299		128,004	96,575	1,062,487	2,477,035			
Nevada	209,920	209,320				410			190	13,433	64,676	131,811			
New Hampshire	1,710,905	1,343,381				8,648	26,800	240,095	91,981	95,555	1,615,350				
New Jersey	11,870,529	8,503,103	5,135,531	3,367,572	55,024	14,470	66,310	2,224,703	1,006,919	240,068	7,456,070	4,174,391			
New Mexico	513,743	490,355	487,133	3,222	2,175	600	7,250		13,363	38,020	317,149	158,574			
New York	28,786,421	25,737,573	18,064,298	7,673,275	59,815	77,418	159,652	2,367,438	384,525	2,166,001	19,581,470	4,311,830		2,727,120	
North Carolina	10 9,400,000									150,000	9,250,000				
North Dakota	1,578,081	1,545,586	1,354,473	191,113		1,528			30,967	280,000	649,041	649,040			
Ohio	9,818,873	1,480,246							8,338,627	(11)	4,909,437	4,909,436		(12)	
Oklahoma	5,515,045									40,000	2,166,018	3,309,027			
Oregon	6,017,759	5,814,502	4,997,783	816,719	26,324	12,513	28,913	70,968	64,539	200,000	4,363,319	1,454,440			
Pennsylvania	24,045,349	18,338,633	12,746,483	5,592,150	34,801	37,444	328,748	1,406,800	3,898,923	133,055,904	15,988,045		5,001,400		
Rhode Island	1,962,898	1,563,067	1,154,236	408,831	754	4,548	13,720	253,260	127,549	175,400	1,787,498				
South Carolina	1,951,559	1,869,961	1,588,972	280,989	16,358	824	24,085	152	40,179	176,560	1,772,180	2,819			
South Dakota	2,429,180	2,401,800	2,106,000	295,800		1,104	26,276			48,584	1,214,590	1,166,006			
Tennessee	3,591,296									214,000	1,688,648	1,688,648			
Texas	14,362,883										10,486,876	3,876,007		576,803	
Utah	634,048										57,245				
Vermont	1,696,582	1,447,679	1,249,047	198,632		4,234	29,497	189,312	25,860	85,955	1,610,627				
Virginia	4,624,475	4,251,509	3,612,282	639,227	4,095	7,422	56,914	38,073	266,462	272,431	4,352,044				
Washington	6,056,003	5,472,663	4,266,572	1,205,091	39,758	14,828	87,619	431,816	9,319	14,649,887	4,548,424	857,692			
West Virginia	3,728,935	3,286,598	2,771,375	515,223	2,381	5,001	52,396	134,075	248,484	15,278,638	1,204,084		2,246,213		
Wisconsin	9,074,490	8,683,997	7,207,564	1,476,433	18,864	91,519	170,450		109,660	549,752	5,214,738	3,310,000			
Wyoming	499,878	489,371	393,899	95,472		858	9,102		547				499,878		
District of Columbia	566,312	120,728	98,986	21,742		1,327	1,906	204,122	238,229	17 228,160					18 338,152
Detailed totals ¹	231,864,509	197,336,270	(19)	(19)	815,216	402,949	1,835,817	11,944,622	19,529,635						
Grand total.....	288,282,352									16,602,211	191,111,302	51,702,184	25,274,158	3,592,497	

¹ The States starred do not show complete receipt details and are not included in totals under first nine receipt columns, shown as detailed total.

² County bond payments marked (c).

³ Includes \$71,828 to probate judges.

⁴ Traffic officers' expenses included.

⁵ Special highway fund taken from certain county registration revenues for special roads included.

⁶ State police expense included.

⁷ For Baltimore streets.

⁸ For expenses of branch offices and motor-vehicle theft department.

⁹ New York City general fund.

¹⁰ Conservative estimate, as data was not received.

¹¹ State appropriation of \$326,265 omitted.

¹² A portion of local road share given to municipal corporations for street repair. Amount not stated.

¹³ State highway patrol expenses of \$491,770 included.

¹⁴ Highway safety fund of \$431,816 included.

¹⁵ Includes \$45,801 refunds.

¹⁶ Includes \$317,011 for administration expenses of State Roads Commission.

¹⁷ Includes \$191,490 for traffic signals.

¹⁸ For repair and construction of Washington streets.

¹⁹ Only 33 States report details of motor-car registration receipts which total as follows: Passenger cars and busses, \$133,584,823; trucks and tractors, \$43,272,495 making a combined total of \$176,857,318.

Tables 5 and 6 show the total number of motor vehicles manufactured, the number of those registered, the number discarded, and the percentage of the new vehicles registered which constitute replacements for the years 1921 to 1926. The motor vehicle production figures have been taken for the year ending September 30, as cars manufactured after this date do not ordinarily appear in the registration figures until the following year. In 1921 only 27 per cent of the new vehicles sold constituted replacements, but in 1926 the figure mounted to 52.2 per cent.

The annual gross receipts from motor-vehicle taxation are given in Table 7. The increase in the revenue from 1920 to 1926 has been at a somewhat greater rate than the registrations, as the average receipts per vehicle have risen from \$11.10 to \$13.10 during the period. The tendency toward increasing motor-vehicle fees is much more evident from 1920 to 1924 than in 1925 and 1926. The increase in the average amount per vehicle is not due entirely to increased license fee charges, but is due in part to a relatively greater

increase in the number of trucks and busses which pay a high license fee.

The disposition of the gross motor vehicle tax receipts for the years 1925 and 1926 is given in Table 8. In both years close to two-thirds of the collections were allocated to State highway department funds, and if payments for interest and in sinking funds for State road bonds are added the amount is increased to nearly three-fourths. In 1926 these combined items amounted to \$211,000,000. The county and other local units received for roads in 1926, including funds for payments on bonds, \$57,000,000, or 19.8 per cent. Collection and administration costs amounted to 5.8 per cent in 1926 but there were nine States which paid these expenses out of other funds. Only 1.3 per cent of the collections were used for miscellaneous purposes. A portion of the amounts included under this item were allotted to general funds of counties including in their area large cities and portions of such funds were used for street work. It is evident that the motor vehicle license tax is now used almost entirely for road purposes.

GASOLINE TAXES FOR THE YEAR 1926

State	Gross tax assessed prior to deduction of refunds	Exemption refunds (deduct from gross tax)	Total tax earnings on fuel for motor vehicles ¹	Collection costs ²	Disposition of total tax earnings			Tax rates, 1926		Date of rate change	Net gallons of gasoline taxed and used by motor vehicles	Estimated additional gallons (not taxed) used by motor vehicles
					Construction and maintenance of rural roads	State and county road-bond payments	Miscellaneous	Cents per gallon				
								State highways	Local roads			
Alabama	\$2,558,651		\$2,558,651	\$9,582		\$2,549,069		2	3 2		127,932,538	
Arizona	1,206,660	\$228,396	978,264		\$489,132	489,132		3	3		32,608,821	
Arkansas	3,785,304	200,000	3,585,304		1,991,834	684,488	\$908,982	4	4		89,632,594	
California	17,910,077	1,407,954	16,502,123		8,251,062	8,251,062		2	2		825,106,169	
Colorado	2,169,456	77,707	2,091,749	(6)	1,045,875	1,045,874		2	2		104,587,460	
Connecticut	2,689,372		2,689,372		2,689,372			2	2		134,468,607	
Delaware	399,309	8,895	390,414		390,414			2	2		19,520,687	
Florida	11,431,486		11,431,486	8,400	8,567,315	2,855,771		4	4		285,787,156	
Georgia	5,653,140		5,653,140	4,200	2,420,974	1,613,983	\$1,613,983	3 1/2	3 1/2		161,518,296	
Idaho	1,182,584	60,367	1,122,217		1,115,397			3	3		37,403,986	
Illinois								0	0	No tax.		650,000,000
Indiana	9,213,828	242,087	8,971,741	11,902	5,973,226	2,986,613		3	3		299,058,025	
Iowa	5,020,086	177,659	4,842,427	10,736	1,610,564	3,221,127		2	2		242,121,370	
Kansas	4,406,653	103,265	4,303,388	(7)	3,576,210	727,178		2	2		215,169,393	
Kentucky	4,935,078		4,935,078	(5)	4,935,078			3	5	Feb. 21	103,477,662	
Louisiana	2,708,567		2,708,567		2,708,567			2	2		135,428,367	
Maine	1,864,596	\$41,250	1,823,346	10,069	1,511,064	302,213		3	3		60,090,659	
Maryland	2,357,577	63,723	2,293,854	2,500	1,833,083		\$458,271	2	2		114,692,672	
Massachusetts								0	0	No tax.		280,000,000
Michigan	10,758,109	676,333	10,081,776	23,737	4,764,422	11,211,557	4,082,060	2	2		504,088,814	
Minnesota	5,072,854	268,166	4,804,688	(12)	4,804,688			2	2		240,234,382	
Mississippi	13 4,088,200		4,088,200	3,150	1,940,186	1,990,912	\$50,727	3	4	Apr. 1	105,887,426	
Missouri	5,777,163	116,018	5,661,145	54,698	5,606,447		\$103,225	2	2		283,057,270	
Montana	870,712		870,712		131,002	477,707		2	15 2		43,535,576	
Nebraska	3,055,705	15,778	3,039,927	7,028	3,032,899			2	2		151,996,357	
Nevada	433,820	28,002	405,818		202,909	202,909		4	4		10,145,454	
New Hampshire	781,453	12,871	768,582		768,582			2	2		38,429,100	
New Jersey								0	0	No tax.		255,000,000
New Mexico	762,851		762,851	25,428	737,423			3	3		25,428,358	
New York								0	0	No tax.		720,000,000
North Carolina	8,113,044	326,571	7,786,473	16 7,786,473		(16)		4	4		194,661,825	
North Dakota	1,083,531	95,038	988,493		820,101		\$168,392	1	2	July 31	73,689,462	
Ohio	13,556,253	298,987	13,257,266		5,965,770	3,314,316	\$3,977,180	2	2		662,863,296	
Oklahoma	6,237,989	18 25,580	6,212,409	(18)	4,141,606	2,070,803		3	3		207,080,296	
Oregon	3,536,142	202,313	3,333,829	7,693	3,326,136			3	3		118,493,937	
Pennsylvania	11 781,782		11,781,782		8,709,213	2,903,071	\$169,498	2	2		588,379,021	
Rhode Island	629,024	117,128	511,896	(21)	511,896			1	1		51,189,641	
South Carolina	4,505,694	8,726	4,496,968	(21)	2,698,181	1,798,787		5	5		89,939,352	
South Dakota	2,284,761	360,003	1,924,758		1,924,758			3	3		64,158,589	
Tennessee	3,852,524		3,852,524	38,525	3,813,998			3	3		128,417,453	
Texas	5,228,000	1,123	5,226,886		3,920,164		\$1,306,721	1	1		522,688,578	
Utah	1,258,009		1,258,009	3,750	1,057,159		197,100	3 1/2	3 1/2		35,943,117	
Vermont	553,093		553,093	(23)	553,093			2	2		27,654,594	
Virginia	6,158,124	302,454	5,855,670	21 697	3,903,316	1,951,657		3	4 1/2	Mar. 11	135,814,061	
Washington	3,701,676	219,583	3,482,093	(25)	3,482,093			2	2		174,104,636	
West Virginia	3,001,131	78,456	2,922,675	(26)	2,922,675			3 1/2	3 1/2		83,504,998	
Wisconsin	5,373,667	163,862	5,209,805	9,982	2,238,574	2,961,250		2	2		260,490,262	
Wyoming	571,449	2,860	568,589		568,589			2 1/2	2 1/2		22,743,572	
District of Columbia	1,020,050	4,857	1,015,193				\$1,015,193	2	2		50,759,671	
Total			187,603,231	238,897	129,441,520	43,609,479	5,238,869	Av.	2.38		7,883,983,560	1,905,000,000

¹ The net tax after deduction of refunds for exemptions according to law and represents the actual taxes available for disposal. The first 2 columns show only the procedure and are not totaled.

² Collection costs in many States are paid from other State funds, and when amounts and sources are reported notes are entered below.

³ Changed to 4 cents on Jan. 4, 1927.

⁴ Allotted by appropriation out of gasoline tax fund, but claims exceed this by \$225,000.

⁵ Collection costs charged to State controllers office fund.

⁶ For State general treasury fund.

⁷ Paid \$8,750 from general revenue of State.

⁸ From general State fund.

⁹ Refund of 2 cents only, allowed by law.

¹⁰ For maintenance of Baltimore Streets.

¹¹ State rewards to counties.

¹² Paid from oil inspection appropriation.

¹³ Includes \$103,225 from extra 2-cent tax collected from Harrison County for sea wall to protect highway in this county.

¹⁴ Sea-wall bonds.

¹⁵ Changed to 3 cents on January 1, 1927.

¹⁶ Large part of State highway share is paid for interest and sinking fund on State highway bonds.

¹⁷ For repair and maintenance of municipal streets.

¹⁸ Deduction of 3 per cent allowed for evaporation.

¹⁹ Paid \$17,616 from State funds.

²⁰ Includes \$14,202 from delinquent 1 cent tax due in 1921-1923.

²¹ State appropriation of \$5,000.

²² For free school fund.

²³ Collection cost of \$500 from motor vehicle bureau appropriation.

²⁴ Only part of cost, remaining cost of \$5,803 is from State appropriation.

²⁵ From Motor vehicle license fund, \$5,000.

²⁶ State appropriation of \$7,500.

²⁷ State road bond payments taken from gasoline tax, amount not reported.

²⁸ For Washington Streets.

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Report of the Chief of the Bureau of Public Roads, 1925.
*Report of the Chief of the Bureau of Public Roads, 1926. 5c.

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Vol. 5, No. 24, D- 6. A New Penetration Needle for Use in Testing Bituminous Materials.
Vol. 6, No. 6, D- 8. Tests of Three Large-Sized Reinforced-Concrete Slabs Under Concentrated Loading.
Vol. 10, No. 5, D-12. Influence of Grading on the Value of Fine Aggregate Used in Portland Cement Concrete Road Construction.
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* Department supply exhausted.

UNITED STATES DEPARTMENT OF AGRICULTURE
BUREAU OF PUBLIC ROADS

STATUS OF FEDERAL AID HIGHWAY CONSTRUCTION

AS OF

MARCH 31, 1927

STATES	FISCAL YEARS 1917-1926				FISCAL YEAR 1927				BALANCE OF FEDERAL AID FUND AVAILABLE FOR NEW PROJECTS		
	PROJECTS COMPLETED PRIOR TO JULY 1, 1926				* PROJECTS UNDER CONSTRUCTION				PROJECTS APPROVED FOR CONSTRUCTION		
	TOTAL COST	FEDERAL AID	MILES		TOTAL COST	FEDERAL AID	MILES		ESTIMATED COST	FEDERAL AID	MILES
Alabama	\$ 18,226,411.34	\$ 8,725,985.09	1,298.3		\$ 6,553,665.00	\$ 3,120,371.51	581.3		\$ 42,206.64	\$ 21,103.32	2.3
Arizona	10,949,878.25	5,863,772.35	723.8		1,481,575.69	874,751.16	81.9		1,323,872.25	664,576.12	50.5
Arkansas	18,364,544.50	7,666,638.35	1,323.0		1,559,719.92	1,559,719.92	206.1				
California	27,142,596.90	12,003,592.30	1,058.0		3,451,010.95	3,451,010.95	162.3		798,158.04	367,739.24	26.8
Colorado	11,905,936.64	5,146,516.16	1,145.0		715,337.14	715,337.14	268.7		472,653.37	234,586.57	1.1
Connecticut	5,414,567.19	2,100,586.80	111.1		584,812.28	245,179.14	65.6				
Delaware	4,918,052.29	1,781,665.60	124.3		1,039,483.07	482,067.18	28.0		283,393.43	129,565.50	14.7
Florida	2,932,660.56	1,664,362.32	134.9		3,372,824.36	1,803,552.23	112.2		957,489.91	359,552.23	24.5
Georgia	24,731,006.97	11,684,727.86	1,794.0		4,555,761.11	2,421,299.11	280.5		779,932.01	392,640.04	2.6
Idaho	11,061,198.14	5,882,112.70	724.7		1,966,337.13	1,075,121.31	103.7		418,539.37	246,879.31	23.0
Illinois	20,619,995.74	1,377.7	534.3		4,022,370.94	1,971,173.14	137.8		4,221,388.28	2,110,694.14	151.1
Indiana	16,959,465.87	8,172,125.19	534.3		7,546,919.61	7,546,919.61	448.8		3,012,000.00	1,501,000.00	106.7
Iowa	29,062,375.40	11,926,302.10	2,114.8		4,503,982.55	2,101,069.80	288.1		4,546,398.59	1,878,372.08	123.2
Kansas	32,866,601.64	12,590,489.25	1,606.6		3,144,505.10	1,487,396.11	247.6		1,239,342.16	483,802.72	80.3
Kentucky	20,737,706.10	8,432,062.25	759.3		2,245,656.73	915,915.55	100.1		1,652,298.13	788,867.96	79.7
Louisiana	13,830,592.68	6,144,739.39	1,055.9		1,528,857.84	691,095.79	77.0		141,920.37	69,698.88	3.6
Maine	8,747,552.76	4,192,507.39	303.6		1,817,247.30	665,945.28	54.0		87,912.01	43,966.00	5.6
Maryland	10,924,943.10	5,112,591.22	423.3		688,351.18	334,258.01	42.4				
Massachusetts	18,353,757.71	6,657,260.62	374.5		936,137.25	332,778.92	10.0		1,333,518.06	377,019.05	24.7
Michigan	25,997,240.78	11,827,052.30	963.0		2,813,653.04	1,245,647.96	74.3		1,983,924.00	943,575.00	51.5
Minnesota	37,170,985.95	15,596,116.56	3,181.9		7,928,662.52	3,430,029.11	481.6		3,029,628.65	786,600.00	131.6
Mississippi	15,146,088.82	7,414,534.10	1,129.0		1,356,914.56	680,646.71	89.5		1,024,955.07	453,130.74	60.3
Missouri	28,993,163.92	13,736,014.85	1,543.2		10,337,573.10	4,559,081.01	310.6		1,760,794.05	776,083.64	97.8
Montana	11,533,401.62	6,333,465.63	1,054.9		1,454,011.91	553,822.80	122.1		881,460.12	473,518.56	101.8
Nebraska	21,624,631.67	5,474,202.52	1,768.3		1,837,751.60	1,837,751.60	395.2		1,197,430.91	582,033.94	55.2
Nevada	7,558,195.61	5,130,934.59	538.8		2,351,683.87	2,351,683.87	300.5		356,783.12	312,536.81	39.4
New Hampshire	4,992,558.60	2,377,450.07	237.6		846,458.09	396,537.45	26.4				
New Jersey	16,346,301.01	5,098,342.21	290.3		5,891,939.07	2,397,022.27	26.0		770,916.35	232,455.00	15.5
New Mexico	12,404,337.77	7,339,657.58	1,427.0		1,951,867.76	1,951,867.76	237.4		495,338.47	389,752.29	7.9
New York	43,224,279.79	17,911,957.19	1,187.0		3,026,682.04	3,026,682.04	136.6		6,432,133.00	1,485,877.50	83.2
North Carolina	27,009,419.47	11,177,377.94	1,259.9		7,603,626.14	3,041,570.47	180.7		938,141.60	360,630.34	20.1
North Dakota	12,313,311.40	6,033,859.78	2,193.1		1,689,601.95	1,689,601.95	536.1		1,273,245.36	636,476.82	84.1
Ohio	67,639,632.90	17,371,787.03	1,364.1		4,552,216.29	1,692,932.11	145.8		825,666.08	276,596.22	16.2
Oklahoma	28,247,850.33	13,159,999.15	1,178.9		1,679,668.40	758,973.69	62.3		3,462,134.01	1,464,462.31	210.1
Oregon	17,637,950.33	8,593,214.79	939.4		1,597,991.40	1,597,991.40	97.1		1,362,864.77	66.4	
Pennsylvania	67,366,150.80	2,960,752.04	1,188.8		10,659,570.21	3,119,354.44	226.8		19,028,021.20	5,499,661.45	379.8
Rhode Island	3,988,616.09	1,558,829.06	86.7		439,650.00	439,650.00	29.3		239,059.07	74,175.00	6.0
South Carolina	15,020,639.90	8,663,322.97	1,481.9		1,873,240.10	711,908.31	75.3		930,746.72	227,507.31	31.9
South Dakota	17,468,373.19	6,603,826.97	2,181.2		785,575.74	785,575.74	261.0		465,885.59	224,609.71	72.2
Tennessee	21,624,631.67	10,276,594.02	780.0		2,470,439.14	1,183,577.64	78.5		111,887.73	35,000.00	3.3
Texas	69,183,673.48	27,440,254.72	4,920.2		7,052,011.25	3,231,149.65	438.4		1,694,198.26	813,238.16	68.7
Utah	8,253,178.03	5,098,440.68	546.4		901,199.30	668,639.27	82.5		129,047.06	91,454.82	11.6
Vermont	4,242,042.64	2,017,639.61	134.5		266,003.78	266,003.78	13.1		1,660,690.47	571,492.05	33.5
Virginia	21,990,249.44	10,386,728.11	1,005.5		3,992,621.16	1,736,445.95	128.4		2,094,951.29	1,280,156.06	60.6
Washington	17,078,511.63	7,782,909.46	683.6		1,105,994.34	463,642.49	42.5		780,099.38	349,827.13	39.7
West Virginia	9,473,716.44	4,141,052.65	392.9		951,130.88	432,685.36	26.5		2,640,353.04	1,392,816.00	71.7
Wisconsin	24,856,508.19	10,392,705.73	1,592.1		2,623,423.79	1,251,529.42	119.9		2,974,796.62	1,392,816.00	16.7
Wyoming	10,928,302.56	6,040,887.05	1,133.5		1,594,423.97	1,016,214.00	172.5		65,821.31	42,256.00	6.6
Hawaii					343,664.16	97,440.00	6.5		346,662.73	98,010.00	6.6
TOTALS	966,692,834.36	426,178,703.58	52,526.6		152,686,512.30	68,889,861.74	7,029.7		56,548,982.02	23,147,846.20	2,023.4
									Federal aid \$ 34,552,347.62	Miles 3,129.6	

* Includes projects reported completed (final vouchers not yet paid) totaling. Estimated cost \$81,673,243.27

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UNITED STATES DEPARTMENT OF AGRICULTURE
BUREAU OF PUBLIC ROADS



VOL. 8, NO. 3



MAY, 1927



GRAVEL ROAD AT ARLINGTON EXPERIMENTAL STATION

PUBLIC ROADS

A JOURNAL OF HIGHWAY RESEARCH

U. S. DEPARTMENT OF AGRICULTURE

BUREAU OF PUBLIC ROADS

CERTIFICATE: By direction of the Secretary of Agriculture, the matter contained herein is published as administrative information and is required for the proper transaction of the public business

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R. E. ROYALL, Editor

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PRINCIPLES OF FINAL SOIL CLASSIFICATION

By Dr. CHARLES TERZAGHI, Massachusetts Institute of Technology, Research Consultant to the Bureau of Public Roads

IF THE foundations of a building settle excessively, thus causing cracking of the walls and ceilings, the reason thereof may reside in excessive compressibility of the soil or lateral flow of the material or both of these factors combined. The speed with which the soil is compressed by the load depends on the permeability of the soil. The more permeable the soil the more rapidly the excess water can escape. Hence, if we wish to describe the properties of a soil, its compressibility and its permeability must be determined and these properties expressed in standard terms.

Cracking of a road surface caused by subsurface conditions may be due to excessive compressibility of the subgrade, to lateral squeezing out of the subgrade, to excessive volume change associated with swelling after a period of drought, to slaking of the material in the shoulders of the road, or to expansion caused by freezing. If the soil is both compressible and permeable the cracking will result essentially from compression of the material caused by pressure. If it is compressible but at the same time very feebly permeable, the failure will be due to lateral flow. The intensity of swelling is simply determined by the degree of elasticity of the material. The slaking properties of the soil depend on both the modulus of swelling and on the permeability. Hence, if we wish to describe the properties of the subgrade of a road which has failed, we have to investigate the factors which influence the amount and the speed of compression of the subgrade under pressure and the volume change due to drying and wetting. These are the outstanding factors in the proposed system of final soil classification. We want to describe our soils not by more or less abstract figures like the Atterberg limits, the dye adsorption number, the moisture equivalent, etc., which are suitable for preliminary purposes only, but rather by data with a direct, simple, and well-known bearing on the behavior of the subgrade.

COMPRESSIBILITY AND ELASTICITY OF SOILS STUDIED

The term "compressibility" refers to the compression produced by a pressure acting on the soil. Suppose we have a layer of fairly loose sand, laterally confined. If the surface of this layer is loaded with 0.5 kilogram per square centimeter (about 0.5 ton per square foot), a measurement of the density of the loaded material may show that the voids occupy a volume equal to 0.97 times the volume occupied by the grains. We say the sand has a voids ratio of 0.97 (ratio between the volume of the voids and the volume occupied by the solids). If the pressure is raised from 0.5 to 3 kilograms per square centimeter, its voids ratio drops from 0.97 to 0.93, which means that the volume of the voids decreases by 4 per cent of the volume occupied by the solids. The voids ratio of a Mississippi clay (fig. 1, A) under a pressure of 0.5 kilogram per square centimeter was found to be equal to 1.67. When the pressure was raised from 0.5 to 3 kilograms per square centimeter the voids ratio became 1.17; in other words, the increase of pressure caused the volume of the voids to decrease by 0.50, representing a decrease in volume equal to 50 per cent of the volume occupied by the solids. Hence we are justified in stating that the clay is exceedingly compressible in comparison with the sand. On the other hand, if the

pressure which acts on the layer of sand is reduced from 3 to 0.5 kilogram per square centimeter, reversing the preceding process, it will be found that the voids ratio increases from 0.93 to 0.94, the increase of the volume of the voids being equal to about 1 per cent of the volume occupied by the solids. The same process of reducing the pressure on a layer of clay caused the voids ratio to increase from 1.17 to 1.30 (13 per cent of the volume occupied by the solid matter). Since the expansion of the soil produced by reducing the pressure represents the reversible part of the preceding compression, we say that the clay is far more elastic than the sand.

The compressibility and the elasticity of a soil can be visualized by plotting a diagram with the pressures as abscissas and the corresponding voids ratios as ordinates. Figure 1 shows such diagrams for eight different typical soils (full-lined curves). For each of these materials the compression test was started at the liquid limit (upper plastic limit), which approximately represents the voids ratio of the soil material when deposited by very slow sedimentation and it has not been subject to any pressure (external or capillary) except its own weight. The pressure acting on the sample was raised in succession from zero to 0.4, 0.8, 1.6, and 3.2 kilograms per square centimeter (approximately) and the voids ratio corresponding to each pressure was determined. With this data the compression curves were plotted. By reducing the pressure from 3.2 down to 1.6, 0.8, 0.4, and zero kilograms per square centimeter (approximately), the soil was allowed to swell, and the points representing the successive states of the expansion determine the swelling curve. From these diagrams it is easy to determine the volume of voids or the water content of the soil standing under a pressure of a given intensity. The steeper the compression curve, the more compressible is the soil, i. e., the greater is the volume change produced by loading the surface of the soil. On the other hand, the steeper the swelling curve, the more the soil will expand as a result of removing an external pressure. As a rule, a steep compression curve is associated with a steep swelling curve.

COMPRESSION OF SOILS REPRODUCED WITH SAND-MICA MIXTURES

The difference between the compression diagrams of the different soils is so striking that it could not fail to attract attention as soon as it was discovered. Since the compressibility of sand with only bulky grains is always, without exception, very small, very much smaller than that of the least compressible material represented in Figure 1, the writer suspected that great compressibility of any soil may be due to the presence of an appreciable percentage of scalelike particles. In order to check this hypothesis, the writer induced G. Gilboy, of the Massachusetts Institute of Technology, to investigate the compressibility and the expansion of differently proportioned mixtures of sand and mica flakes, both with a grain size of 0.5 millimeter. The results of these investigations were striking. By properly selecting the mica content of the sand it was possible to imitate both the compressibility and the elastic rebound of all the clay soils which thus far have been investigated. In Figure 1 the dotted lines represent the effect of raising the pressure up to 3 kilo-

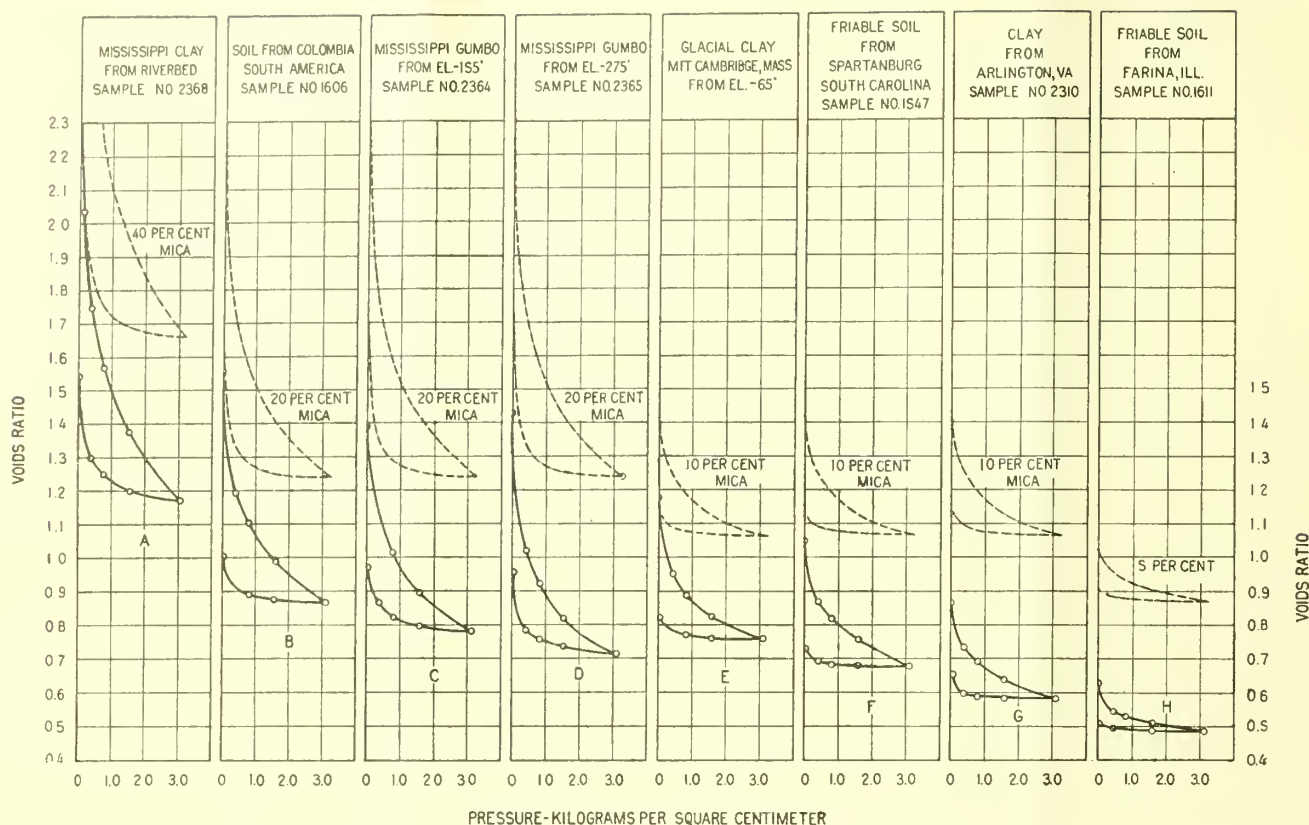


FIG. 1. —TYPICAL TIME-COMPRESSION CURVES FOR SOIL SAMPLES AND MIXTURES OF SAND AND MICA

grams per square centimeters and subsequently reducing it to zero on the particular sand-mica mixture which seemed to bear the closest resemblance to the soil whose compression curve is shown below. The soil curves are lower in the diagram than the sand-mica curves only because of the difference in uniformity of the two materials. As a rule, the more uniform a material is the greater is its volume of voids in a loose state.

Another difference between the curves representing soil and sand-mica is the range over which the two materials expand if completely relieved from pressure. Although the shapes of the expansion curves for both materials may be almost identical, the final expansion (due to removing the last pressure increment) is always very much smaller for the clays, than for the equivalent sand-mica mixtures. This is because the sand-mica mixture is perfectly cohesionless, the particles being free to move, while the clay always retains part of its cohesion, even after complete removal of the external pressure. (Fig. 1.)

It should be noticed that the voids ratio at the beginning of each test is in every case less for the clay soils than for the corresponding perfectly cohesionless sand-mica mixture. Since the initial voids ratio of the clay was always equal to or slightly above the liquid limit, we learn from Figure 1 that the liquid limit corresponds to at least as dense and as stable a mass as that of a well-shaken sand-mica mixture.

Figures 2 and 3 illustrate the difference in compressibility of various sand-mica mixtures. The upper row of pictures in Figure 2 show different mixtures, each weighing 200 grams and with mica contents of 0, 5, 10, 20, and 40 per cent, respectively. The lower row of pictures show the compression produced by a standard pressure of 1 ton per square foot.

Thus it became evident that the compressibility and elastic rebound of soils has very little to do with effective size, uniformity or colloid content. The controlling factor seems to be merely the mechanical effect of a greater or smaller abundance of scalelike particles.¹ In the final soil classification, which is on the point of being worked out, full advantage will be taken of this important fact.

GRAIN-SIZE CONTROLS SPEED OF COMPRESSION

The only difference between the results of compression tests performed on a clay soil and on an equivalent sand-mica mixture concerns the speed with which the decrease in volume takes place. Suppose the sand-mica mixture came to rest under a pressure

¹ This hypothesis was merely based on the similarity between the elastic properties of clays and of sand-mica mixtures. Facts however accumulate which seem to show that the clays really do contain a considerable percentage of scalelike particles. Some of them were published in the writer's book, *Erdbaumechanik*, Vienna 1925, which discusses plasticity of the powders of minerals which break up into scalelike particles, difference between the linear shrinkage of clay samples in the direction of the pressure exerted on the samples in pressing them into molds and the linear shrinkage perpendicular to this direction, etc. Quite recently C. S. Ross and E. V. Shannon succeeded in determining the physical form of the clay minerals of bentonite and related clays and found the following constituents: Montmorillonite (micaceous), beidellite (micaceous), micaceous halloysite (micaceous), halloysite (amorphous or submicroscopically crystalline), kaolinite (platy crystalline). "The minerals of bentonite and related clays and their physical properties," by C. S. Ross and E. V. Shannon. *Journal of the American Ceramic Society*, vol. 9, February, 1926. Thus out of five clay minerals contained in bentonite, three are micaceous, one platy crystalline, and only one is apparently amorphous. The elasticity (elastic rebound) of average clays corresponds to the elasticity of a sand-mica mixture with 10 per cent mica and those of the most elastic clays to mixtures with 20 per cent mica. Hence the findings of Ross and Shannon represent an important step towards explaining the similarity between clays and sand-mica mixtures.

There exists, however, one more group of soil constituents, which seem to have an effect on the elasticity of the soils very similar to the effect of mica. This group consists of the organic colloids. One year ago the writer showed, that the compression curve of gelatine is almost the same as the compression curve of an accumulation of mica flakes. The difference between the gelatine and the mica powder merely consists in the fact that the gelatine is almost perfectly elastic the expansion curve being almost identical with the compression curve, whereas the mica powder is imperfectly elastic, as are all accumulations of individual grains. From what we know thus far, a high content in organic matter associated with mica content (silts) causes considerable elasticity and an abnormally high liquid limit combined with a comparatively low plasticity index.

of 1.5 kilograms per square centimeter. If the pressure is increased rapidly from 1.5 to 3 kilograms per square centimeter, from 90 to 95 per cent of the compression takes place almost at once. Then the compression slowly increases as time goes on and after a couple of hours, equilibrium is practically reached, and the compression proceeds no further. Figure 4, A shows the results of time observations made on a layer of a 20 per cent sand-mica mixture. When making the same test on a clay with equal compressibility, practically none of the compression takes place at once, the time required for obtaining 75 per cent of the total compression depending on both the permeability of the material and the thickness of the layer. Figure 4, B shows the time-compression curve for a highly plastic soil from Colombia, South America, and Figure 4, C shows an equally plastic gumbo from the bed of the Mississippi River. The compressibility of these three materials is shown in Figure 1, A, B, and C. Since all three materials are practically equally compressible and elastic, it seems logical to associate the differences between the time-compression curves with differences in grain-size of the materials.

If the mechanics of the process of consolidation under pressure is considered, it at once becomes obvious that a low degree of permeability must be associated with slow consolidation. Suppose a 20-centimeter layer of a material with the compressibility of the soils in Figure 4 be subject to a rapid increase in pressure from 1.5 to 3.1 kilograms per square centimeter, resulting in a decrease in thickness of 1.4 centimeters. If the voids of the material are filled with air only, the compression of the layer from 20 to 18.6 centimeters can take place at once, except for the time required for the particles to adjust themselves to the modified conditions of stress within the layer. In cases like that of Figure 4, A, the lag is insignificant. On the other hand, if the voids of the material are filled with water, compression from a thickness of 20 centimeters down to 18.6 centimeters requires a sheet of water 1.4 centimeters thick to flow from the interior of the material through its upper surface. No water can flow unless there is a hydraulic gradient, and the less permeable the material, the slower will be the flow under a given gradient. Hence, the time required to press a sheet of water 1.4 centimeters thick out of the material will depend on how permeable the material is.

The mechanics of the lag in compression can best be learned by discussing an apparatus as illustrated in Figure 5. Figure 5, A, represents a cylindrical vessel with a perforated piston the weight of which is compensated by the pressure exerted by two springs, S_1 and S_2 . If the space below the piston is empty, a pressure applied on top of the piston will cause the piston to move almost instantaneously downward through a distance, d , depending on the compressibility of the springs. On the other hand, if the space below the piston is filled with water, the downward movement of the piston will be retarded on account of the time required for the water to escape through the openings of the piston. The smaller the openings, the longer it will take the piston to reach its final position. During the downward movement, the water located beneath the piston will be under a pressure the intensity of which depends on the hydraulic gradient required to force the water through the openings. The longer the pressure acts, the lower the speed with which the water escapes, and finally the system will come to

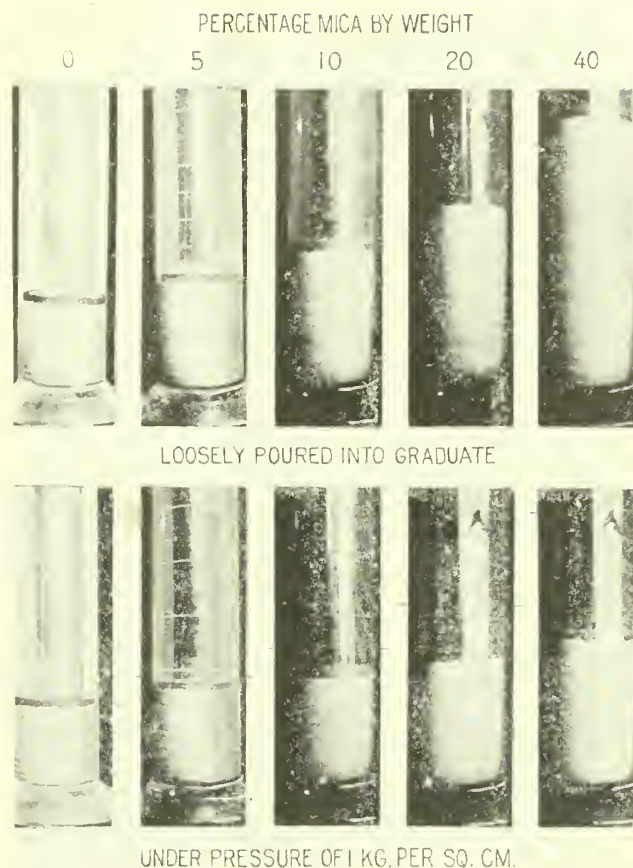


FIG. 2.—ACTUAL SAMPLES OF SAND-MICA MIXTURES SHOWN DIAGRAMMATICALLY IN FIGURE 3

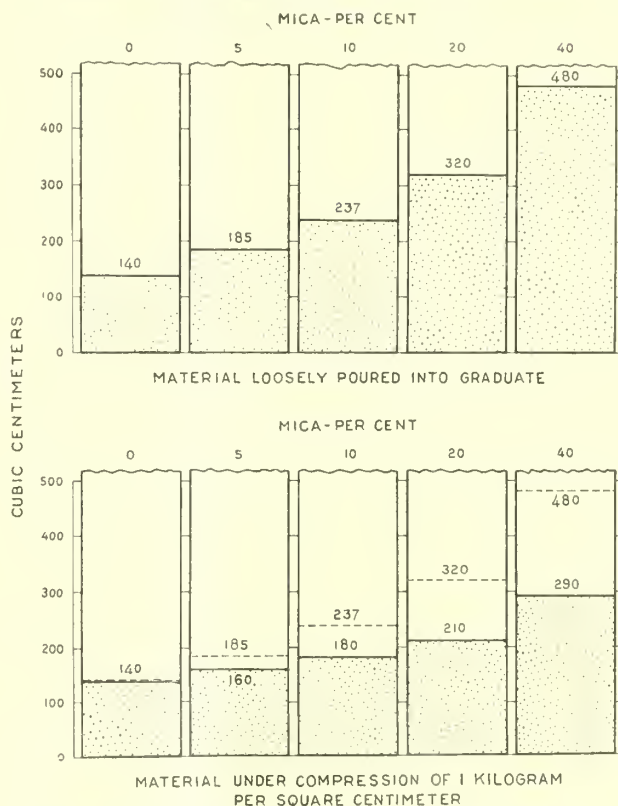


FIG. 3.—COMPRESSION PRODUCED BY A PRESSURE OF 1 KG. PER SQ. CM. ON 200 GM. SAMPLES OF VARIOUS SAND-MICA MIXTURES

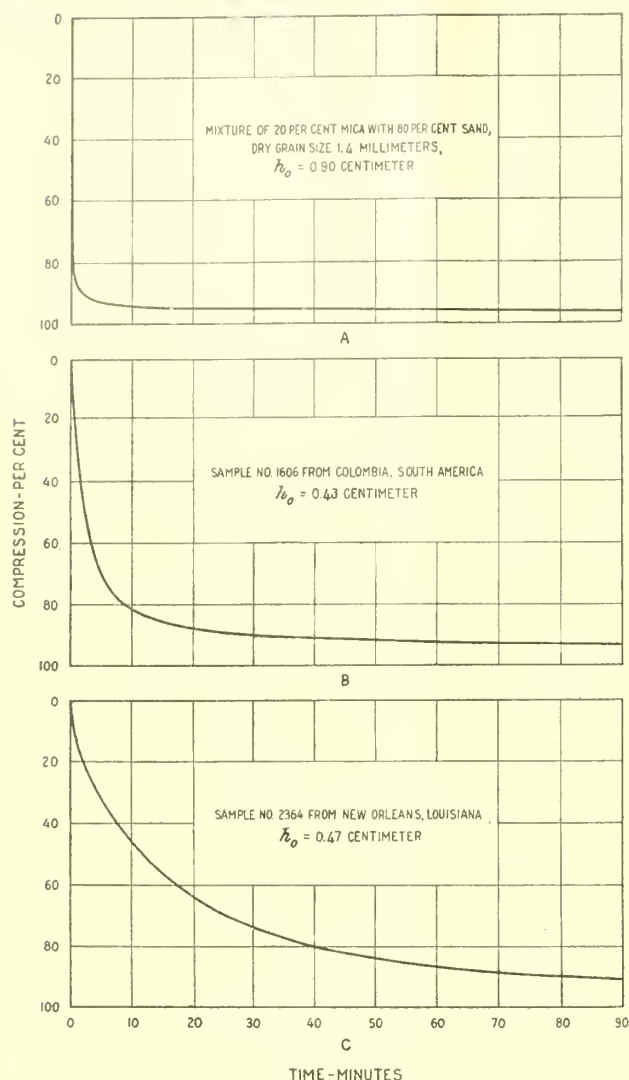


FIG. 4.—COMPARISON OF TIME-COMPRESSION CURVES FOR A 20 PER CENT SAND-MICA MIXTURE AND TWO SOILS. COMPRESSION PRODUCED IN 17 HOURS BY RAISING PRESSURE FROM 1.58 TO 3.10 KG. PER SQ. CM. IS TAKEN AS 100 PER CENT

rest and the hydrostatic pressure in the water beneath the piston becomes equal to zero.

The water contained in the voids of a water-soaked, compressible layer of soil (fig. 5, B) behaves under pressure in precisely the same manner as does the water beneath the piston of Figure 5, A. The only difference is that in the case of Figure 5, B, the obstacles which interfere with the escape of the water are scattered throughout the space; hence, the hydrostatic excess pressure will be different in different parts of the layer. Nevertheless, the size of the openings still remains the principal factor controlling the speed with which the water escapes.

MATHEMATICAL RELATION EXISTS BETWEEN PERMEABILITY AND SPEED OF COMPRESSION

Since the speed of the process of consolidation depends on nothing but the quantity of water to be forced out of the soil and on the resistance against flow of the water through the material, it has been possible to work out a theory of consolidation of water-soaked soils under pressure and to compute the permeability from the speed with which consolidation

proceeds. Since this theory has been published elsewhere² the results only are presented here. Let

h_0 be the reduced thickness of the layer of soil in centimeters, i. e., the thickness the layer would have at a volume of voids equal to zero.

p_0 the pressure acting on the sample before the beginning of the test in grams per square centimeter.

$p_0 + p_1$ the pressure acting on the sample during the test, in grams per square centimeter.

t the elapsed time in minutes after the pressure was raised from p_0 to $p_0 + p_1$.

e_0 the voids ratio of the sample at the beginning of the test.

e the voids ratio of the sample after time t .

e_1 the voids ratio of the sample after an infinite time.

$\Delta e = e_0 - e$ the compression after a time t .

$\Delta e_1 = e_0 - e_1$ the final compression produced by increasing the pressure from p_0 to $p_0 + p_1$.

(Since the voids of the soil are supposed to be completely filled with water, the voids ratio, e , is synonymous with the water content in cubic centimeters per cubic centimeter of solid soil material.)

$a = \frac{\Delta e_1}{p_1}$ (p_1 in grams) the coefficient of compressibility, i. e. the average decrease in voids ratio produced per gram of additional pressure.

Finally, the speed, with which the process of consolidation proceeds depends on the value of k , the coefficient of permeability. To explain the meaning of the coefficient of permeability, let us suppose a layer of soil with a thickness, h , inclosed between two vertical screens. (Fig. 6, A.) Through each of these screens the voids of the soil are supposed to communicate with bodies of free water. If the water on the left side of the soil stands higher than on the right side, the water percolates through the soil from the left to the right. According to Darcy's law the speed of percolation increases in direct proportion with the hydraulic gradient (ratio between the difference in water pressure on both sides of the layer of soil and the distance through which the water percolates, which in this case, is equal to the thickness, h , of the layer). If the difference in elevation of the two water levels is equal to h , the thickness of the layer, the hydraulic gradient is equal to unity. The speed of percolation at a hydraulic gradient equal to unity is called the coefficient of permeability k_1 . It should, however, be noticed, that the speed of percolation does not indicate the velocity with which the water moves through the voids of the soil, but the speed with which it moves immediately after it leaves the soil, through the unobstructed space adjoining the downstream screen. Hence the speed of percolation expresses as well the quantity of water which passes per unit of time through a unit of area of the layer.

What precedes represents the current definition of the coefficient of permeability. It refers to the true thickness of the layer of soil. In contrast to this, in the computations concerning the consolidation of soils we are obliged to refer everything to the reduced dimensions of the samples; that means to the thickness the samples would have if the space occupied by the voids were reduced to zero. Otherwise it would be impossible mathematically to solve the equations. The reduced thickness of the sample has been called h_0 . The true thickness, h , is equal to $h_0 (1 + e)$ wherein e is the voids ratio.

² TERZAGHI, CHARLES. DIE BERECHNUNG DER DURCHLÄSSIGKEITZIFFER DER TÖNE AUS DEM VERLAUF DER HYDRODYNAMISCHEN SPANNUNGSERSCHINUNGEN. Sitzungsberichte der Akademie der Wissenschaften in Vienna, mathem. naturw. Klasse, Abt. II a, pp. 125-138, 1923. A short abstract appeared in Engineering News-Record, Nov. 26, 1925.

The reduced coefficient of permeability, k , is the speed with which the water would percolate through the sample if the difference between the two water levels were h_0 instead of h . This speed of percolation is obviously smaller than the true coefficient of permeability. Since, according to Darcy's law, there exists

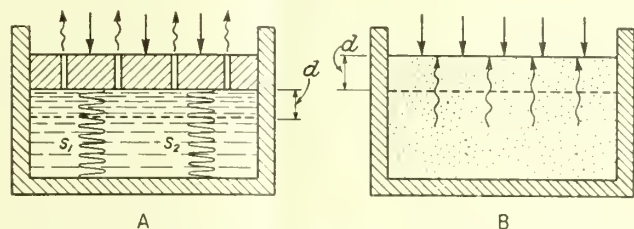


FIG. 5.—APPARATUS ILLUSTRATING ACTION OF WATER UNDER HYDROSTATIC PRESSURE, ESCAPING THROUGH SMALL OPENINGS

a simple proportionality between the hydraulic gradient and the speed of percolation, we can write

$$k_1 : k = \frac{h}{h_0} : \frac{h_0}{h} = 1 : \frac{1}{1+e}$$

$$\text{Hence } k = \frac{k_1}{1+e}$$

When making the standard compression test, the water percolates through the sample—not in a horizontal direction, as shown in Figure 6, but in a vertical direction. After the pressure has been raised from p_0 to $p_0 + p_1$, the voids ratio decreases from e_0 to $e_1 = e_0 - \Delta e_1$. The average voids ratio during this process is $e = \frac{e_0 + e_1}{2} = e_0 - \frac{\Delta e_1}{2}$. In introducing this value of e into our formula we obtain for the reduced coefficient of permeability the formula

$$k = \frac{k_1}{1 + e_0 - \frac{1}{2}\Delta e_1}$$

While the voids ratio of the sample decreases from e_0 to e_1 , both the true and the reduced coefficient of permeability decrease. At the same time the value of the coefficient of compressibility decreases, the aforementioned value, a , merely representing the average for the whole range from e_0 to e_1 . Fortunately the fundamental equations contain neither the value of k nor the value, a , individually, but they contain the ratio between these two values, viz, $c = \frac{k}{a}$. This value is called the coefficient of consolidation. While both the values of a and of k rapidly decrease with the voids ratio, the variations of c are far less significant, which fact considerably increases the degree of accuracy of our computations.

To make the list of symbols used in the following equations complete, we have, as a result of the preceding discussions, to add

k_1 the coefficient of permeability of the soil in cm. per min., or the velocity with which the water flows through the soil under a hydraulic gradient equal to one,

$k = k_1 \frac{1}{1 + e_0 - \frac{1}{2}\Delta e_1}$ the reduced coefficient of permeability, and

$c = \frac{k}{a}$ the coefficient of consolidation.

By theory it was found that the aforementioned quantities are connected with each other by the following equation:

$$\Delta e = \Delta e_1 \left[1 - \frac{8}{\pi^2} e^{-\frac{c\pi^2}{4h_0^2}t} - \frac{8}{9\pi^2} e^{-\frac{9c\pi^2}{4h_0^2}t} - \dots - \frac{8}{(2n+1)^2\pi^2} e^{-\frac{(2n+1)^2c\pi^2}{4h_0^2}t} \right] \quad (1)$$

Figure 7 shows this relation graphically, for a material with $\Delta e_1 = 100$ per cent and $\frac{c\pi^2}{4h_0^2} = 1$. The effect of the

assumption $\frac{c\pi^2}{4h_0^2} = 1$ will be discussed later. In Figure 8 the results of time observations with gelatine have been plotted. The small circles represent the results of the measurements and the full line shows the theoretical consolidation curve according to the preceding equation. Since the internal friction in gelatine is very small, the agreement between theory and observation is almost perfect.

PERMEABILITY COMPUTED FROM TIME OBSERVATIONS

When a layer of water-soaked soil consolidates under pressure, the time effect due to low permeability combines with the time effect due to internal friction. According to equation (1) the time effect due to low permeability increases in direct proportion with the square of the reduced thickness h_0 of the layer. In contrast to this, the time effect due to internal friction (gradual adjustment of the soil particles) was found to be practically independent of the value of h_0 . Hence interpretation of the time-compression curves representing test results is apt to be misleading unless one succeeds in separating the two effects from one another. This can be done on account of the fact that if there were no appreciable resistance against the escape of the water, the first part of the compression would occur almost instantaneously.

In the following discussion this part will be called $n\Delta e_1$, the value of the coefficient, n , ranging according to the type of the soil and to the value of p_0 anywhere between 0.6 and 1 (60 and 100 per cent). As a rule, the value of n is about 0.85 (more or less). The second part $(1-n)\Delta e_1$ represents the compression due to gradual adjustment of the grains and the speed with which this part of the compression takes place, is essentially governed by internal friction (friction between the soil grains). Hence if there is an appreciable lag between $\Delta e = 0$ and $\Delta e = n\Delta e_1$ this lag is almost exclusively due to the resistance against the flow of the water. The shape of the curve for the lag due to low permeability is very characteristic. Due to this fact the problem merely consists in finding out how far the time-compression curve follows the law expressed by equation 1 and in determining the con-

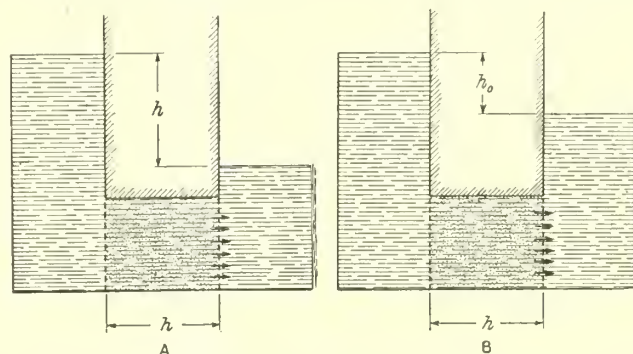


FIG. 6.—DIAGRAMS USED IN DEFINING MEANING OF k_1 THE COEFFICIENT OF PERMEABILITY AND k THE REDUCED COEFFICIENT OF PERMEABILITY

solidation curve follows the law expressed by equation 1 and in determining the con-

stants of this first section of the curve. To solve this problem a simple graphical method has been devised.

This method is based on the following facts: Let

Δe be the compression produced by increasing the pressure from p_0 to $p_0 + p_1$, after a time t_1 (fig. 7),
 $\Delta e_1 = 100$ per cent compression, the compression after an infinite time, and
 t_2 the time required to produce the compression $\frac{\Delta e}{2}$ (fig. 7).

If the values of Δe are plotted as ordinates and the corresponding values of $\frac{t_1}{t_2}$ as abscissas (curve C in fig.

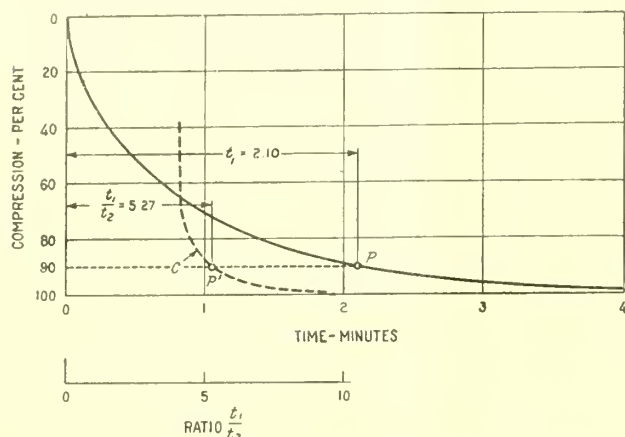


FIG. 7.—THEORETICAL TIME-COMPRESSION CURVE

7) a curve is obtained with a very characteristic shape.

From $\Delta e = 0$ up to $\Delta e = 0.7\Delta e_1$ the value of $\frac{t_1}{t_2}$ is approximately equal to 4, which means that within this range, the consolidation curve is approximately a parabola.

Between $\Delta e = 0.7\Delta e_1$ and $\Delta e = 0.94\Delta e_1$ the value of $\frac{t_1}{t_2}$ increases from 4 to 6, and between the limits of $\Delta e = 0.94\Delta e_1$ and $1.00\Delta e_1$ it goes rapidly up from 6 to infinity.

When comparing theoretical curves of the type of Figure 7 with the actual time-compression curves for a considerable number of soils the following fact was noticed. As a rule, the empirical curves have very nearly the same shape as the theoretical curves up to a point which, on the theoretical curve, has an ordinate of 90 per cent (approximately). Beyond this point the discrepancy between the two curves increases very rapidly, in such a manner, that the 100 per cent ordinate of the theoretical curve corresponds to an ordinate n equal to anywhere between 80 and 100 per cent on the actual curve. An ordinate of 90 per cent ($\Delta e = 0.90\Delta e_1$) on the theoretical curve corresponds to a value of $\frac{t_1}{t_2}$ of 5.27.

Hence, in order to adapt equation 1 to an empirical time-compression curve (see, for example, the curves of fig. 9, soil No. 2250), we start by tracing a short section of the $\frac{t_1}{t_2}$ curve (dotted curve C in fig. 9, B to E) and determine on this curve the point P_1 for which the value of $\frac{t_1}{t_2}$ is equal to 5.27. Then we find the point P on the consolidation curve whose ordinate is equal to the ordinate of P_1 . The point P corresponds to the point P on the consolidation curve, Figure 7, and also to the time t_1 in the consolidation diagrams, Figure 9.

If there were no internal friction in the soil, the consolidation would occur according to a theoretical curve

which passes through the point P and which approaches a horizontal asymptote with an ordinate, n , equal to $\frac{1}{0.90}$ times the ordinate of the point P . The problem merely consists in determining the value of c for which the theoretical curve passes through the point P with an abscissa t_1 and an ordinate 90 or 90 per cent.

In order to solve this problem we return to equation 1. In Figure 7 the point P with an ordinate 90 corresponds to an abscissa $t_1 = 2.1$. If we introduce in equation 1 the data for point P in Figure 7, viz,

$\frac{c\pi^2}{4h_0^2} = 1$ and $t_1 = 2.1$, we obtain:

$$\Delta e = \Delta e_1 \left[1 - \frac{8}{\pi^2} e^{-1 \times 2.1} - \frac{8}{9\pi^2} e^{-9 \times 1 \times 2.1} - \dots - \frac{8}{(2n+1)^2 \pi^2} e^{-(2n+1)^2 \times 2.1} \right] = 0.9\Delta e_1.$$

If we had plotted our curve (fig. 7) for any other value of $\frac{c\pi^2}{4h_0^2}$, for instance, $\frac{c\pi^2}{4h_0^2} = 5$, we would have obtained

for t_1 the value $t_1 = \frac{2.1}{5} = 0.42$ simply because our equation does not furnish the result $\Delta e = 0.9\Delta e_1$, except for one definite value of the exponent of e . Hence, if for $\frac{c\pi^2}{4h_0^2}$ we assume a value of 5 instead of 1 then the value of t_1 would become $\frac{2.1}{5}$, and we have

$$-(2n+1)^2 \times 1 \times 2.1 = -(2n+1)^2 \times 5 \times \frac{2.1}{5} =, \text{etc.}$$

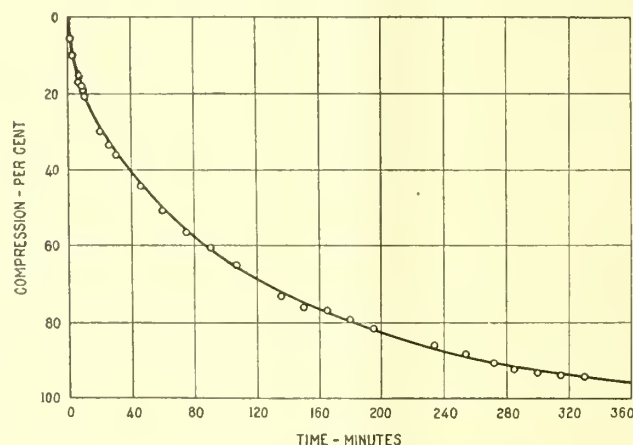


FIG. 8.—FULL LINE REPRESENTS THEORETICAL CURVE WHILE CIRCLES SHOW CLOSE AGREEMENT OF TEST ON ISOELECTRIC GELATINE, LOWERING THE PRESSURE FROM 11.1 TO 5.57 KG. PER SQ. CM. THE TIME OF TEST WAS ONE WEEK. THE TEST WAS ACTUALLY MADE BY ALLOWING THE GELATINE TO EXPAND RATHER THAN COMPRESSING IT, BUT SINCE GELATINE IS ALMOST PERFECTLY ELASTIC THERE IS PRACTICALLY NO DIFFERENCE BETWEEN COMPRESSION AND EXPANSION CURVES

In Figure 9 the abscissa t_1 is determined by the test results. Since, according to what precedes, for $\Delta e = 0.9\Delta e_1$, the product $\frac{c\pi^2}{4h_0^2} t_1$ must be equal to 2.1, we can write the equation

$$\frac{c\pi^2}{4h_0^2} t_1 = 2.1$$

or the factor of consolidation of our sample

$$c = \frac{2.1 \times 4}{\pi^2} \times \frac{h_0^2}{t_1} = 0.85 \frac{h_0^2}{t_1} \dots \dots \dots (2)$$

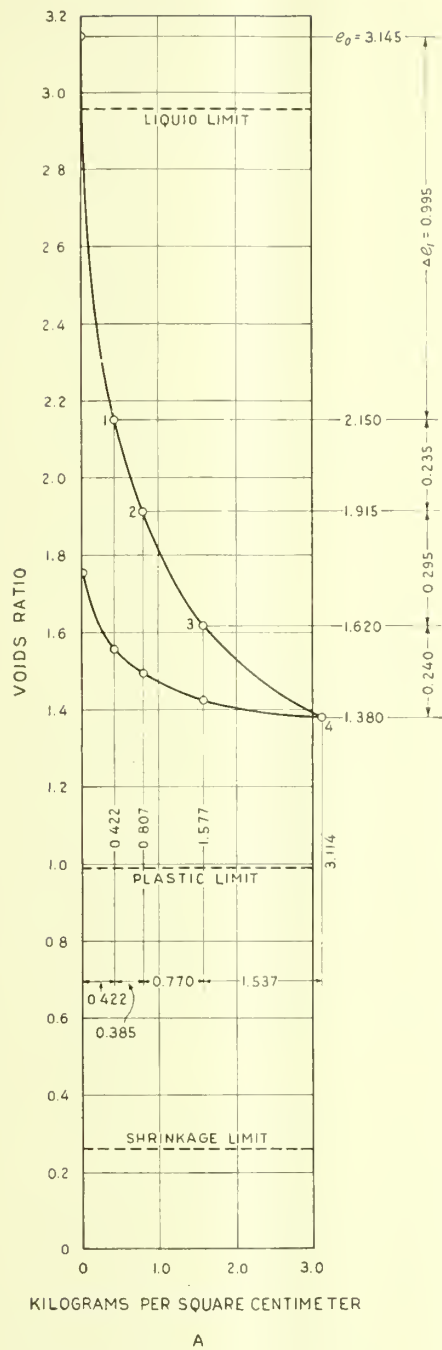
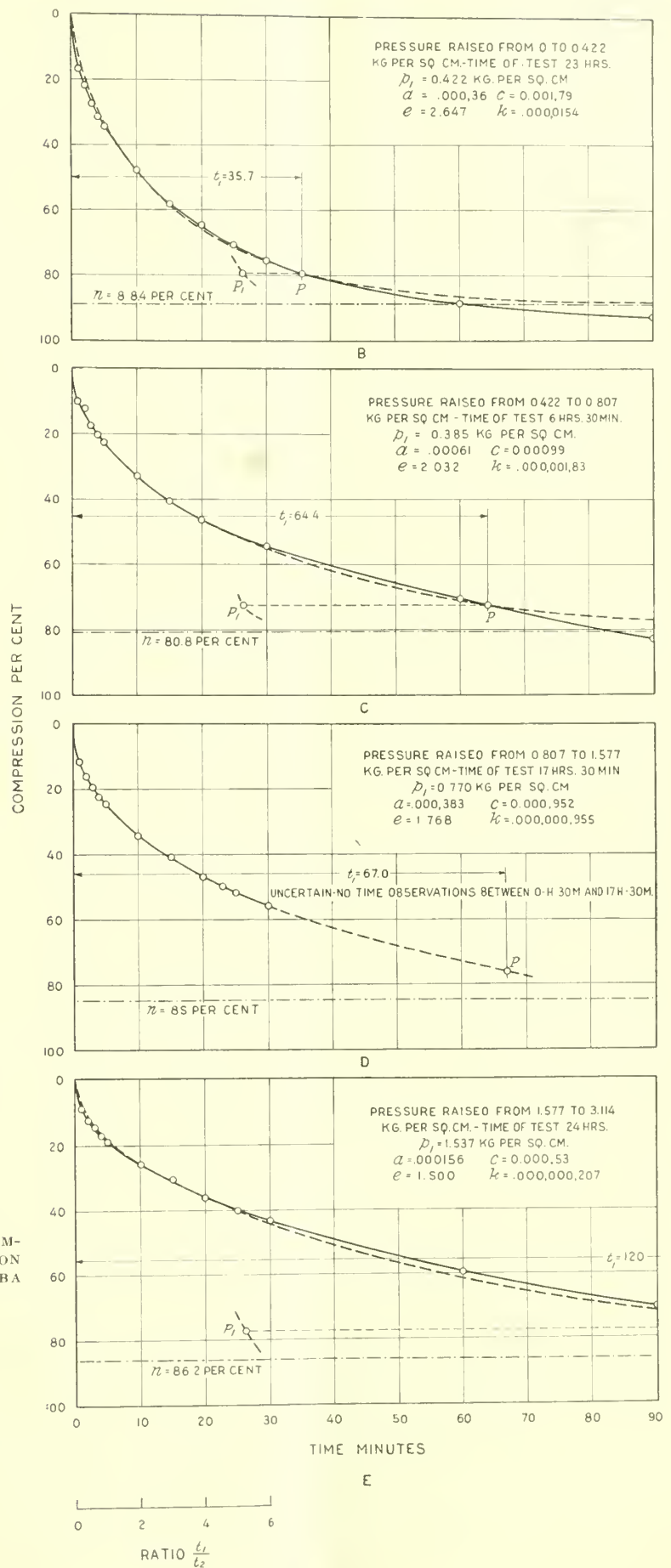


FIG. 9.—Load-compression curve and time-compression curves as a result of tests on sample No. 2250, YAGUAJAY CLAY FROM CUBA



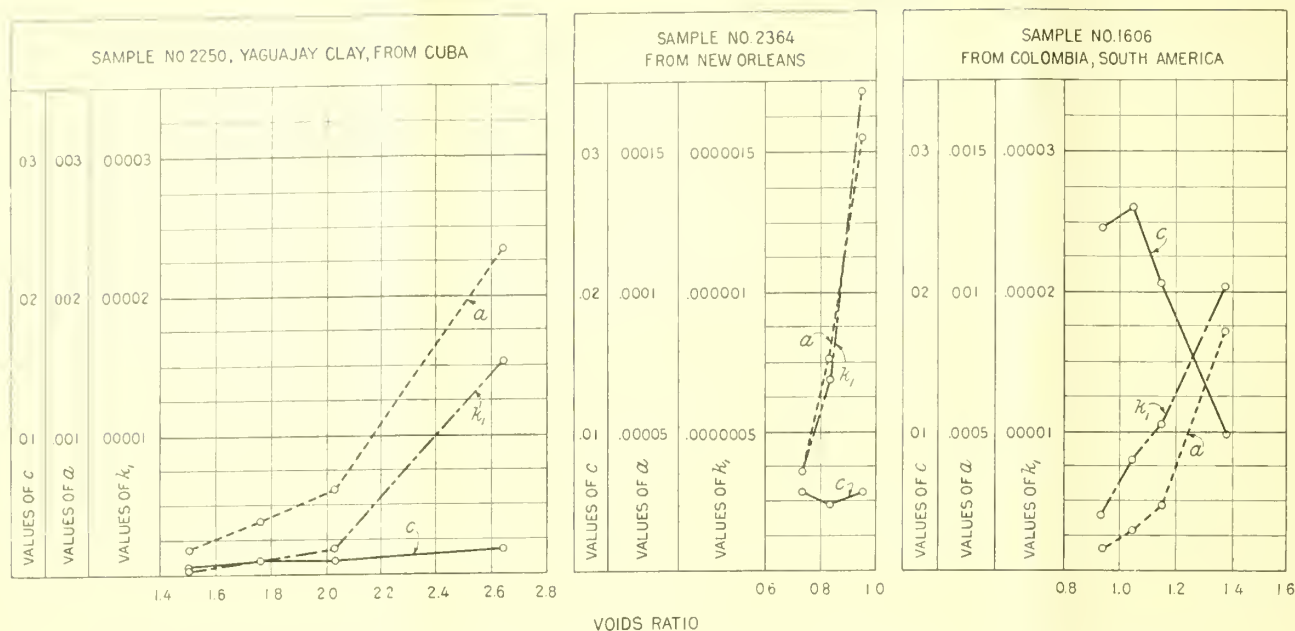


FIG. 10.—CURVES SHOWING VARIATION IN VALUE OF c , a , AND k_1 , FOR THREE TYPICAL SOIL SAMPLES

wherein h_0 is the reduced thickness of our sample in centimeters (see nomenclature for equation 1) and t_1 is the time in minutes which corresponds to point P on the consolidation curve.

The value of c being determined, it remains to consider how to find the value of k_1 (coefficient of permeability). Strictly speaking, the value c derived from any time-compression curve represents the average coefficient of consolidation for the range e_0 to $e_0 - n\Delta e_1$. In addition to this, the following fact should be kept in mind: According to Figure 9, the lag due to low permeability occurs, as if the compression of the sample were equal to $n\Delta e_1$, the corresponding coefficient of compressibility being equal to $\frac{n\Delta e_1}{p_1}$ instead of $\frac{\Delta e_1}{p_1}$.

However, considering the errors involved in the graphical procedure as such and the degree of accuracy required for classification purposes, these facts are disregarded, except when making investigations for scientific purposes. For current work we refer all the values resulting from the computation for any specific pressure interval p_0 to $p_0 + p_1$ to the voids ratio $e_0 - \frac{1}{2}\Delta e_1$ (average voids ratio for the time-compression process) and consider the coefficient of compressibility equal to $\frac{\Delta e_1}{p_1}$. On this assumption we obtain

$$a = \frac{\Delta e_1}{p_1}, \quad k = ca, \quad \text{and} \quad k_1 = ca(1 + e_0 - \frac{1}{2}\Delta e_1).$$

METHOD ILLUSTRATED BY EXAMPLE

As an example of what precedes, Figures 9 and 10 may serve. Figure 9, A, shows the pressure-voids ratio curve for a highly plastic clay from Cuba and the Figures 9, B to E, the results of the time-pressure observations. The pressure was raised by increments from zero to 0.422, 0.807, 1.577 and 3.114 kilograms per square centimeter. The corresponding values of p_1 and Δe_1 can be learned from Figure 9, A. They are as well represented in Table 1. This table shows in addition the numerical computations in the order in

which they follow each other. The data for computing the values contained in the lines 1 to 5 of the table are obtained from the pressure-voids ratio diagram Figure 9, A. The next step consists in determining the value t_1 . For this purpose one first traces a smooth curve through those points in the time-compression diagram, which represent the actual observations. In the diagrams, this curve is shown by a full line. Then one plots the curve C , whose abscissas represent the values of t_1 . With some skill one usually succeeds in obtaining the information required by plotting not more than three points on this curve. The point whose abscissa is $t_1 = 5.27$ represents point P_1 . The abscissa of the point P which corresponds to the point P_1 on the t_1 -curve gives the value t_1 in the table.

For the purpose of checking the actual observations against the theoretical values determined by equation 1 the following procedure can be followed: In Figure 7 the ordinates 0.2, 0.4, 0.6, 0.8, 0.90, and 0.95 (20, 40, etc., if expressed as a percentage) correspond to the abscissas (values of t_1) 0.08, 0.316, 0.70, 1.42, 2.10, and 2.80. In the time-compression diagram (fig. 9), the ordinate of the horizontal asymptote to the theoretical curve is equal to n , equal to $\frac{1}{0.90}$ times the ordinate of point P . Hence, in order to trace the theoretical curve through the point P in Figure 9, divide the space between zero and n into equal parts, such as to obtain on the vertical axis points with the ordinates $0.2n$, $0.4n$, $0.6n$, $0.8n$, and $0.95n$. Since the difference between the theoretical curves in Figure 7 and Figure 9 is one of scale only, the abscissas of P being equal to 2.1 and t_1 , respectively, the abscissas of the theoretical curve in Figure 9, for the ordinates $0.2n$, $0.4n$, $0.6n$, $0.8n$, $0.90n$ and $0.95n$ will obviously be equal to $t_1 \frac{0.08}{2.1}$, $t_1 \frac{0.316}{2.1}$, $t_1 \frac{0.70}{2.1}$, $t_1 \frac{1.42}{2.1}$, $t_1 \frac{2.80}{2.1}$. In the time-compression diagrams Figure 9, B to E, the theoretical curves thus obtained are shown by dotted lines.

TABLE 1.—*Computation of coefficients of consolidation and permeability for sample No. 2250 for four increments of load with $h_0=0.274$ cm.*

	First load increment	Second load increment	Third load increment	Fourth load increment
p_1 in grams per square centimeter	422	385	770	1,537
e_0	3.145	2.150	1.915	1.620
Δe_1	0.995	0.235	0.295	0.240
$e = e_0 - \frac{\Delta e_1}{2}$	2.647	2.032	1.768	1.500
$a = \frac{\Delta e_1}{p_1}$	0.00236	0.00061	0.000383	0.000156
t_1 in minutes	35.7	64.4	67.0	120.0
n	0.88	0.81	0.85	0.86
$c = 0.85 \frac{h_0^2}{t_1} = \frac{0.0638}{t_1}$	0.00179	0.00099	0.00095	0.00053
$k_1 = ca(1+e)$	0.0000154	0.0000183	0.0000096	0.0000021

After the values of t_1 have been found, the values of c and k_1 can be computed (lines 8 and 9 in Table 1). Figure 10, A shows the results of the computations graphically, the values of a , c and k_1 plotted against voids ratio. Figure 10, B and C, represent similar test results for two other soils.

These diagrams bring out the known fact that the coefficient of permeability decreases very rapidly with decreasing voids ratio. The same is true for the coefficient of compressibility. Coefficients as variable as these are certainly not suitable for classification purposes. On the other hand, the degree of permeability of a soil is a property of such far-reaching and universal importance that no classification for engineering purposes can be satisfactory without considering it. The simplest way to overcome the difficulty is to express it indirectly, by the average coefficient of consolidation. Figure 10 indicates that the coefficient of consolidation is far less variable than either a or k , and at the same time it depends directly on permeability. The coefficient of consolidation of soils varies from less than 0.001 for very plastic clays to 500 or more for very fine sands. Hence no great accuracy in computing and expressing this coefficient is required for characterizing the soils.

It is proposed to make not more than two sets of time observations for every standard volume-compression test, one for raising the pressure from 0.4 to 0.8 kilogram per square centimeter approximately and one from 1.5 to 3 kilograms per square centimeter approximately and to consider the average coefficient of consolidation to be equal to the mean of these two values, omitting all decimals. The coefficient of consolidation for the first pressure increment from zero to 0.4 kilogram per square centimeter is disregarded, because this coefficient has been found to be usually an abnormal one, being excessively low or excessively high (fig. 10, A and B). Thus, for the three soils represented in Figure 10, the coefficients of consolidation would be equal to one-half $(0.00099 + 0.00053) = 0.0008$ for sample No. 2250, 0.005 for sample No. 2364, and 0.02 for sample No. 1606, these figures representing the order of magnitude of the coefficients.

COEFFICIENT OF CONSOLIDATION IS OF PRACTICAL SIGNIFICANCE

It is believed that the coefficient of consolidation is of considerable practical significance. Suppose we have two equally compressible clay soils, A and B, soil A with a coefficient of consolidation of 0.001, and soil B with a coefficient of consolidation of 0.1. If a similar load is placed on top of each of these soils the compression will be produced one hundred times more

rapidly in soil B than in soil A. Or, if the surface of both soils be simultaneously exposed to the air, so that the water can gradually evaporate, the dry crust which forms on soil B will be very much thicker than the dry crust on soil A. Finally, if both soils come into contact with open water, after having previously dried out, the soil B will swell one hundred times more rapidly than will the soil A.

It has been indicated that the value of the coefficient of consolidation ranges between extraordinarily wide limits. Since this coefficient determines the speed with which a pressure is apt to compress the soil, provided its voids are filled with water, it ranks among the most characteristic constants of the soil.

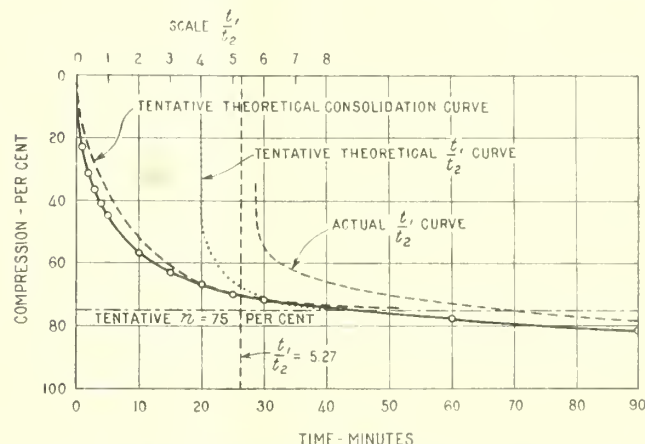


FIG. 11.—AN ABNORMAL CONSOLIDATION CURVE SHOWING CONSIDERABLE DEPARTURE FROM THEORETICAL CURVE. CURVE REPRESENTS COMPRESSION ON RAISING PRESSURE FROM 1.577 TO 3.114 KG. PER SQ. CM.

For friable soils consolidation is apt to proceed so rapidly that the time observations can not be made with the same degree of precision as can be done for slowly consolidating materials. In that case, a simple modification of the test arrangement makes it possible to obtain the data concerning the permeability of the material independent of the time-compression observations. The modification consists in compressing the material by means of a piston with a permeable bottom instead of using a piston with a watertight bearing surface. The permeability of the sample is computed from the speed with which the water flows from a narrow standpipe through the sample and through the permeable bottom of the piston into the hollow interior of the piston, while the sample is under pressure. The test results combined with the data furnished by the compression observations determine the value of the coefficient of consolidation.

The same procedure should be followed in those cases where the consolidation curve has an abnormal shape, that means where the value of n is smaller than approximately 0.8, a condition which seems to be quite common with feebly compressible soils with a low coefficient of permeability. In these cases the shape of the entire curve may considerably differ from the normal shape of the consolidation curve, as shown in Figure 11. This abnormality seems to be due to the finer fraction expanding into the larger voids while consolidation proceeds. The interpretation of the time-compression curve obtained under such conditions leads to values which are too high for the coefficients of consolidation and of permeability.

Hence, in order to determine the coefficient of consolidation two different procedures should be followed, depending on the character of the consolidation curve. If n is equal to or greater than approximately 0.80, and if in addition the value of t_1 for $\Delta e = 0.70 \Delta e_1$, is greater than five minutes, the coefficient of consolidation can be computed from the consolidation curve. Otherwise, the coefficient should be determined from the results furnished by the modified test arrangement.

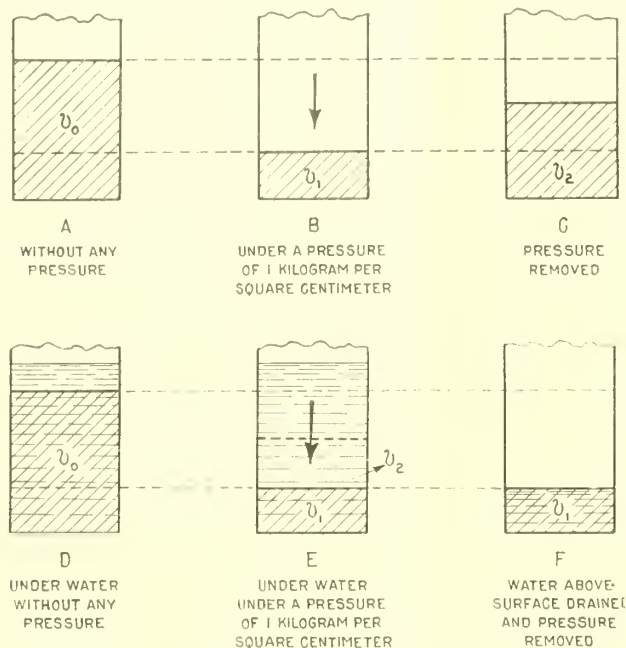


FIG. 12.—VOLUME CHANGE OF COTTON UNDER ORDINARY AND CAPILLARY PRESSURE

ACTION OF CAPILLARY PRESSURE ON POROUS MATERIAL INVESTIGATED

Thus far, we have merely considered the volume change of the soils (compression or expansion) produced by a change of the external pressure. It is known, however, that fine-grained soils such as clays and loams may change their volume apparently without any external pressure acting upon them. It is sufficient to expose them to the air to make them shrink, and subsequent wetting causes them to expand (swell). Until a short time ago it was generally assumed that shrinking and swelling were due to somewhat inexplicable properties of the colloids contained in fine-grained soils. Recent investigations, however, seem to show that the causes of shrinkage and swelling are very much simpler and that they reside in the action of no other forces than those which drive the water up in capillary tubes or in similar narrow channels. In order to visualize the action of these forces, let us first consider the behavior of a piece of cotton or of a very fine-grained sponge, subjected to an external pressure. Suppose cotton or sponge is put into a vessel (fig 12, A) with no load acting on top of it. For this state the material occupies a certain volume V_0 regardless of whether it is dry or completely immersed in water. Under the influence of a pressure of, say, 1 kilogram per square centimeter the volume goes down from V_0 to V_1 (fig. 12, B) again regardless of whether the sample is dry or completely immersed. We say that we have compressed the material. Finally, if the

pressure is removed, the volume goes up from V_1 to V_2 , smaller than V_0 (fig. 12, C). The material has expanded, the expansion being due merely to the elasticity of the substance. Since V_2 is smaller than V_0 , we say the material is imperfectly elastic, $V_2 - V_1$ representing the elastic volume-change and $V_0 - V_2$ the inelastic or permanent volume-change produced by the external pressure.

The elastic expansion from V_1 to V_2 takes place regardless of whether the material is dry or completely immersed; i. e., we will get practically the same result regardless of whether the test is performed on the dry sample or on the same sample completely saturated with and covered by water. Yet, if prior to the removal of the external pressure, all the water standing above the surface of the sample is drained off and the pressure is removed (fig. 12, F), then the material fails to expand. Its volume remains practically unchanged. From the observation made during the first stage of the test (fig. 12, A to C) it is known that it requires a pressure of 1 kilogram per square centimeter to keep the material confined within a space V_1 . As far as the test conditions are concerned, the only difference between the two cases, represented by Figure 12, C, and F, is that in the first case the surface of the sample is completely covered with water while in the second the surface of the sample represents a boundary between air and water. Hence one is compelled to assume that the force which replaced the external pressure, thus preventing the material in the case of Figure 12, F from expanding, resides within the boundary between sample, air and water.

It remains to investigate how that force comes into existence. For this purpose let us consider a vessel simply filled with water. If a steel needle with a somewhat greasy surface is carefully placed on the water surface, the needle floats (fig. 13), whereby it produces a slight depression. Since the specific gravity of the needle is very much greater than the specific gravity of the water, this phenomenon would not be possible unless the water surface behaved like a thin rubber skin with a small, but nevertheless very definite tensile strength. This tensile strength has repeatedly been measured. It amounts to 75 dynes (equal to 0.0764 gram) per centimeter. This value is known as the surface tension of the water.

ACTION OF CAPILLARY PRESSURE ON A COMPRESSIBLE TUBE

The tensile strength of the top layer of the water or the surface tension of the water comes still more into prominence if the lower end of a capillary tube is



FIG. 13.—CAPILLARY FORCE SUPPORTING A GREASY NEEDLE ON WATER SURFACE

dipped into a vessel filled with water. If only a very short piece of the tube sticks out of the water, the water at once shoots up to the upper end of the pipe and stays there. It is obvious that the water could not stay there unless it were held by some force, equal to the weight of the body of water located between the surfaces of the water in the tube and in the vessel. This force is nothing but the tension in the top layer, acting like a rubber skin and tied to the walls of the

tube by molecular attraction. At a constant temperature the surface tension of the water remains always the same (0.0764 gram per centimeter). Since the circumference of the tube is equal to $2\pi r$, the direction of the force acts at an angle α with the vertical and the specific gravity of the water is equal to one, equilibrium requires (fig. 14, A)

$$\pi r^2 y = 0.0764 \times 2\pi r \cos \alpha$$

If the tube is gradually pulled out of the water, the weight of the water to be carried by the surface tension

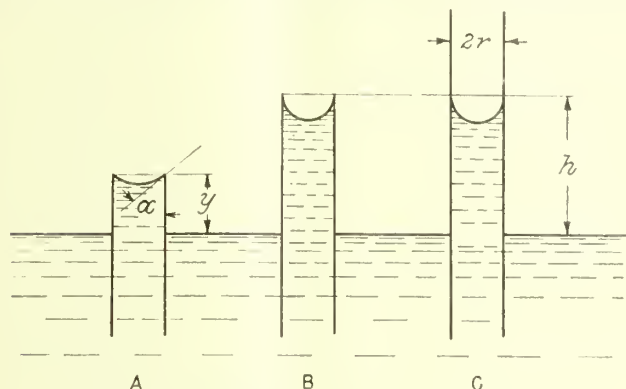


FIG. 14.—WATER IN CAPILLARY TUBES

increases. The surface tension remains the same. Hence, equilibrium requires the value of the angle α to decrease until finally α becomes zero. (Fig. 14, B.) This event marks the maximum height to which the water can possibly rise within the tube. Introducing $y=h$ and $\alpha=0$ into the formula we obtain

$$\pi r^2 h = 0.0764 \times 2\pi r$$

$$\text{or } h = \frac{0.0764 \times 2}{r} = \frac{0.1528}{r} \quad (3)$$

The smaller the radius, r , of the capillary tube the greater is the height, h , to which the water may possibly rise. If the tube is pulled still further out of the water after the height, h , is reached the meniscus (curved water surface within the tube) remains within the tube, at a constant elevation, h , above the outside water level. (Fig. 14, C.)

In order to connect these phenomena with Figure 12, let us consider a capillary tube made out of a very compressible material, such as rubber. At the outset (fig. 15) the tube is supposed to be completely filled with water and the water surfaces at both ends of the tube are practically flat. If the water evaporates, the volume occupied by the water becomes smaller. Yet the same force which drove the water surface up in the tubes (fig. 14) now prevents the water from retiring into the interior of the tubes. (Fig. 15.) Thus, as a result of the evaporation of the water, the surface tension starts to act. The plane water surface of Figure 15, A, passes into a curved one (fig. 15, B) and the axial component of the surface tension produces an equal pressure in the wall of the tube, which in turn causes the compressible wall of the tube to become shorter. This force is equal to $0.0764 \times 2\pi r \cos \alpha$ grams. The more water evaporates, the smaller becomes the angle α . With a decreasing angle α the compressive force increases, until finally, for $\alpha=0$, the force becomes a maximum, corresponding to a maximum shortening of the tube. If, in this or in any

preceding state of the evaporation process the tube is placed under water, the free water surface disappears. With the disappearance of this surface the force acting on the tube disappears, and as a consequence the tube expands, its length increasing again from l_3 to the original value l_1 , provided the tube be perfectly elastic. If the tube is imperfectly elastic, the expansion is smaller than the preceding contraction.

CAPILLARY PRESSURE ON SOILS CALCULATED

The pressure exerted by the surface tension of the water on the wall of the tube may be called the capillary pressure. Capillary pressure fully explains the behavior of the porous sample represented in Figure 12, because it is nothing but a system of compressible capillary tubes whose upper openings are located within the top surface of the sample. By draining the water off the surface of the sample (fig. 12, F) a free water surface is formed within the top surface of the sample, and whereby the pressure of 1 kilogram per square centimeter is still acting. In this state the water surface is still a plane. However, the very moment the pressure is removed the surface tension of the water starts to act. The surface stretching across each individual opening becomes a curve, the angle α assuming such a value that the total force exerted by the capillary pressure becomes equal to the pressure which has been removed, thus keeping the volume of the sample unchanged and equal to V_1 . (Fig. 12, F.)

The replacement of the external pressure by an equal capillary pressure obviously requires that the maximum value of the capillary pressure be equal to or greater than the external pressure which it replaces. Hence it is interesting to figure out how great the force exerted by the capillary pressure may be. For this purpose, suppose we have a sample whose voids have an average width of 0.01 millimeter, which corresponds to the size of voids of a very coarse silt. Assuming the voids are square, every opening would have a perimeter of

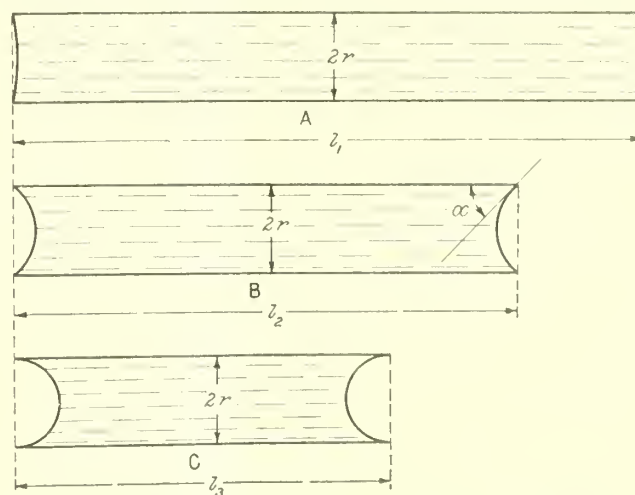


FIG. 15.—SHORTENING OF COMPRESSIBLE TUBES DUE TO CAPILLARY FORCE

0.04 millimeter = 0.004 centimeter, and the force acting on the rim of every opening will be equal to

$$0.004 \times 0.0764 = 0.000306 \text{ grams}$$

This is the force acting on the area of one opening, and as the water extends continuously throughout the whole specimen a force of the same intensity acts over the

entire surface regardless of the actual number of openings per unit of area. The force per square centimeter of surface is therefore the force per opening multiplied by the number of openings per square centimeter, assuming the surface to be completely covered by such openings or

$$\frac{0.000306}{0.001^2} = 306 \text{ grams per square centimeter}$$

In algebraic terms, the computation would be as follows: If the voids of a powder are assumed to be square, the average length of the sides of the square being equal to a in centimeters, the surface tension acting on the rim of each square is equal to

$$4a = 0.0764 = 0.306a \text{ grams}$$

The area of each opening is equal to a^2 . Hence the maximum value of the capillary pressure will be equal to

$$\frac{0.306a}{a^2} = \frac{0.306}{a} \text{ grams per square centimeter} \quad (4)$$

SOIL SHRINKAGE DUE TO CAPILLARY PRESSURE

From the preceding it is obvious that shrinkage is nothing more than compression produced by a gradual increase of the intensity of the capillary pressure, comparable to the shrinkage of the compressible tube of Figure 15. Swelling represents elastic expansion of the material caused by reducing the capillary pressure to zero. Considering in addition the fact expressed by equation 4, the relation which exists between compressibility, grain size, and volume change due to shrinkage can easily be deduced. For this purpose let us consider a powder or a soil with the compressibility of sample No. 1606 of Figure 1, B. According to this figure, such a soil is approximately as compressible and as elastic as a coarse-grained mixture of 20 per cent mica and 80 per cent sand, because compressibility and elasticity have nothing to do with grain size.

But now, let us take two members of this class, one with an average width of voids of 0.1 mm. and the other with a width of voids of 0.001 mm., mix them with water, so that the water content approximately corresponds to the liquid limit, and then let the water evaporate. As soon as the water starts to evaporate, the surface tension of the water comes into effect, and in both materials the capillary pressure will increase, until it reaches its maximum value. According to equation 4, the maximum value of the capillary pressure will, for the coarse-grained soil, be equal to

$$\frac{0.306}{0.01} = 30.6 \text{ grams per sq. cm.}$$

while for the fine-grained one, it will be as high as

$$\frac{0.306}{0.0001} = 3060 \text{ grams per sq. cm.}$$

To find the volume change produced by these capillary pressures, it is necessary to consult Figure 1. If both materials are assumed to have precisely the same compression curve as sample No. 1606, their initial voids ratio would be 1.56. Under the influence of the capillary pressure, the voids ratio of the coarse-grained material would decrease from 1.56 down to

$$1.45, \text{ corresponding to a volume change of } \frac{1.56 - 1.45}{0.0256} =$$

4.3 per cent of the original volume, while the voids

ratio of the finer one would decrease to 0.87, corresponding to a shrinkage of $\frac{1.56 - 0.87}{0.0256} = 27$ per cent of the

original volume. Hence, it is clear that for materials with equal compressibility, the shrinkage increases with decreasing grain size. The shrinkage of the soil shown

in Figure 1, C was found to be equal to $\frac{1.56 - 0.53}{0.0256} =$

40.2 per cent. (0.53 equals shrinkage limit expressed by voids ratio), which indicates that the voids of this material were considerably narrower than 0.001 mm.

On the other hand, for materials with different compressibility but with equal grain size, the maximum values of the capillary pressure will be equal, but the shrinkage will be different. Suppose we have two soils, one of the type of sample No. 1606 of Figure 1, B and one of the friable type like sample No. 1641 of Figure 1, H. If the width of the voids of both materials is equal to 0.001 mm. the maximum value of the capillary pressure will be equal to 3,060 grams per sq. cm. Yet, the volume shrinkage of the soil of

Figure 1, B would be equal to $\frac{1.56 - 0.87}{0.0256} = 27$ per cent

against $\frac{0.626 - 0.485}{0.04626} = 8.68$ per cent for the soil of

Figure 1, H.

The swelling caused by immersing a stiff, plastic or dry piece of soil is determined by the shape of the swelling curves in Figure 1. The steeper these curves, the more intense is the volume change associated with wetting. Hence the shape of the swelling curve serves as an index of the intensity of the swelling of the material.

The action of the surface tension of the water being explained, there remain only two phenomena associated with the shrinking and the swelling of fine-grained soils which can not be traced back to simple capillarity, and which required elaborate investigations before their nature was understood. These two phenomena are the behavior of fine-grained soils upon drying to a water content beyond the shrinkage limit, and the fact that certain fine-grained soils retain part of their cohesion, if they are immersed after having previously been compacted by external or by capillary pressure.

"SOLIDIFIED" WATER BINDS FINE-GRAINED SOILS

If in Figure 15, C, the water continues to evaporate, the meniscus retires into the interior of the tube, the column of water becomes shorter and shorter, and finally, after the water has completely disappeared, the tube has expanded just as completely as if it had been immersed. If we pass from the tube to the powder, one would expect that complete evaporation would cause the powder to expand, and at the same time to lose its cohesion. As a matter of fact, if a very fine-grained sand, having originally some cohesion caused by the presence of water in the voids, is dried, it becomes again perfectly cohesionless. In contrast to this, the cohesion of very fine-grained soils increases upon drying, instead of disappearing. The cause of this phenomenon was found to reside in the fact that water contained in very narrow fissures (smaller than one ten-thousandth of a millimeter) has no longer the physical properties of normal water. It has a higher viscosity, a greater surface tension, and, what seems to be most important of all, it does not evaporate even

if it is heated to the boiling point of normal water.³ Within such openings the water is in what may be called a solidified state. In very fine-grained soils a considerable percentage of the capillary water is confined to openings smaller than the size mentioned above. Hence, when drying the soil at 100° C., the "solidified" part of the water remains and acts similarly to a cement, binding the grains together.

The capacity of certain fine-grained soils to retain part of their cohesion after immersion in water is due to actual cohesion between the soil particles. It has been found that any two bodies, if brought into contact, stick to each other.⁴ The force with which the bodies adhere is so small that under ordinary circumstances it can not be measured. It is even less possible actually to observe it. However, if the number of particles which touch each other is very great, then the adhesion acting at every point of contact becomes evident. Suppose the adhesion per point of contact to be equal to 0.0001 gram. If two pieces of gravel adhere to each other with that force the presence of the force would obviously escape our attention. But if we have an accumulation of soil particles with an average diameter of 0.01 millimeter (size of coarse silt particles), each one of them adhering to its neighbors with a force of 0.0001 gram, the shearing strength of the accumulation would be equal to $1,000^2 \times 0.0001 = 100$ grams per sq. cm. = 0.1 kg. per sq. cm., a shearing strength which involves quite a considerable compressive strength.

CONSISTENCY OF SOILS IN DRY AND SATURATED STATE TO BE DETERMINED

If a fine-grained soil whose original water content was equal to or somewhat above the liquid limit dries out it passes in succession from the liquid through the plastic and the semisolid states into the solid state.⁵ During this process the material becomes stiffer and stiffer, owing to the gradual increase of the intensity of the capillary pressure. We say the material becomes more and more consistent, wherein the term "consistency" indicates the resistance of the material to flow. Methods for determining the consistency have been discussed in a preceding paper.⁶ There remains to consider the states for which the consistency of soils should be determined for the purpose of soil classification.

The extreme states between which the consistency of a subsoil may vary are the dry state and the state of utmost saturation owing to immersion of the soil after a period of drought. The consistency of the soil in the second state seems to have an important bearing on both the bearing capacity of the soil after continued rainfalls and on the slaking properties of the soil. Hence, it was decided to include in the program for final soil tests a compression test on an unconfined, cylindrical sample of the dried soil and a consistency test on a soil sample which has completely expanded, under no pressure, after it has been previously compressed under a pressure of approximately 3.1 kilograms per square centimeter. In order to obtain further information concerning the effect of the type of soil on the consistency in different states, some consistency tests are to be made on soil samples which have

been compressed under a pressure of approximately 3.1 kilograms per square centimeter, without being allowed to expand prior to the test. The consistency tests are simply compression tests performed under standardized conditions on cylindrical, unconfined samples with a diameter of 1 inch.

It is obvious that any change in water content of a soil located at or near the surface of the ground causes the soil to crack. When the soil samples are prepared for consistency tests, the samples have no chance to crack. Hence the results of the consistency tests furnish information only on the consistency of the individual pieces, which, in their totality, would form an "undisturbed" subgrade. Yet, for the purpose of soil classification, such information can be considered sufficient.

CONCLUSIONS

The data on which it is proposed that the final soil classifications be based, give information about the following properties of the subgrade:

1. The volume change produced by a change of the external pressure (load) which acts on the soil (compressibility and elasticity of the soil, represented by compression and swelling curves of the type of Figure 1).
2. The speed with which the volume change follows a change of the pressure (coefficient of consolidation, derived from the results of the time observations by means of equation 2).
3. The permeability of the soil (coefficient of permeability, computed from the coefficient of consolidation or, for more permeable soils, directly obtained by means of a permeability test, performed under standard conditions).
4. Volume change due to drying and wetting, under standard conditions (obtained from the shrinkage limit and from pressure—voids-ratio diagrams of the type of Figure 1).
5. Consistency of the soil in two extreme states.

The technic of the tests required for obtaining the aforementioned data will be described in another paper. All this data has a simple and well-defined bearing on the behavior of the subgrade under load and under variable atmospheric conditions.

The investigations concerning the colloidal character of soil constituents, dye adsorption, base exchange, etc., fall in the same class as the recent investigations concerning the effect of the carbon content, and of various alloys on the strength of steel, or analyses of the physical and chemical action in cement during the process of setting, or similar intricate physicochemical problems relating to construction materials. There is no question of the value of the ultimate results of such investigations. Yet in the classification of construction materials for engineering purposes it is very doubtful whether anything more efficient than the present system, based upon the behavior of the materials under stress, will ever be obtained.

If a road surface cracks because of subsurface conditions, or if a foundation settles, it is due exclusively to a strain in the subgrade, produced by a change of the intensity and distribution of the pressure which acts in the subgrade. Hence, there is no reason to base the final system of soil classification on anything except on the behavior of the soil under various conditions of stress and of confinement. Since the data required for classifying the soils are obtained from actual measurements, performed under standardized conditions, the principles of the system will continue to be valid regardless of the ultimate outcome of physical and chemical investigation of soils.

³ TERZAGHI, CHARLES. VERSUCHE ÜBER DIE VISKOSITÄT DES WASSERS IN SEHR-ENGEN DURCHGANGSQUERSCHNITTEN. Zeitschrift für angewandte Mathematik und Mechanik. Band 4, S. 107-113, 1924.

— THE MECHANICS OF ADSORPTION AND THE SWELLING OF GELS. Monograph of the Fourth National Colloid Symposium. 1926.

⁴ TERZAGHI, CHARLES. ERDBAUMECHANIK. Pp. 64-66. Vienna, 1925.

⁵ TERZAGHI, CHARLES. SIMPLIFIED SOIL TESTS FOR SUBGRADES AND THEIR PHYSICAL SIGNIFICANCE. PUBLIC ROADS, vol. 7, no. 8, p. 153, Oct., 1926.

⁶ TERZAGHI, CHARLES. DETERMINATION OF THE CONSISTENCY OF SOILS BY PENETRATION TESTS. PUBLIC ROADS, vol. 7, no. 12, Feb., 1927.

ANALYSIS OF STRESSES IN CONCRETE ROADS CAUSED BY VARIATIONS OF TEMPERATURE

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THIS PAPER supplements a previous paper by the writer, published in *PUBLIC ROADS*, April, 1926, under the title "Stresses in concrete pavements computed by theoretical analysis," and in the Proceedings of the Fifth Annual Meeting of the Highway Research Board, held at Washington, D. C., December 3-4, 1925, Part I, 1926, pp. 90-118, under the title "Computation of stresses in concrete roads." Like the previous paper, it rests upon the assumption that the concrete pavement acts as a homogeneous elastic solid, and, as in the former analysis, the method is that of the theory of elasticity, conclusions being drawn from a few simple physical laws by mathematical analysis. In regard to the general assumptions and concepts the reader is referred to the previous paper, which dealt with stresses and deflections produced by wheel loads. The present discussion deals with stresses and deflections produced by variations of temperature. The former paper, on account of the complexity of the mathematical processes involved, was limited to a statement of the assumptions and general principles and to the presentation of the results with illustration of how to use them. Since the mathematical processes involved in the present paper are much simpler, it was considered to be expedient not to omit them. Results are given in tables and diagrams, and the use of the results is illustrated by numerical examples.

The fact that cracks sometimes develop in a new pavement before any load is put on it shows the importance of the stresses which ordinarily are considered to be secondary—stresses caused by changes of temperature, by setting of the concrete, and by changes of moisture content. In a piece of concrete free to expand and contract the setting will produce a shrinkage equal to that caused by a certain drop in temperature, and the absorption of moisture will produce a swelling equal to that produced by a certain rise in temperature. In each case the essential feature is a tendency to change of volume. Thus, the study of stresses due to variations of temperature will supply information also about the other secondary stresses. With this condition in view, the discussion is limited to the case of variations of temperature.

The stresses caused by variations of temperature should be considered from two points of view. One has reference to the early life of the pavement, before the pavement has obtained its full strength and before it has been opened to traffic. The other has reference to the later life, after the pavement has gained strength and after it has been opened to traffic. According to the first point of view, the stresses caused by variations of temperature are considered as an independent cause of cracks and are treated by themselves. According to the other point of view, the temperature stresses are to be combined with the stresses caused by the loads. This combination, in most cases, is a simple matter of addition.

Whatever particular effect is studied, three places on the panel of pavement should be examined—the corners, the interior area, and the edges.

Two major cases will be investigated. The first arises from a consideration of the slow seasonal changes of temperature; the second from a consideration of quick changes of temperature, occurring, for example, by the change from a cool night to a hot day, and vice versa. In the first case the final temperature is assumed to be uniform throughout the thickness of the pavement. The second case is dealt with by assuming a definite temperature gradient through the thickness of the pavement with the temperature unchanged at the middle plane. The stresses in this case are dependent upon the manner in which the slab deflects or curls. Obviously, the stresses found in these two major cases may have to be combined by addition or subtraction.

STRESSES CAUSED BY UNIFORM DECREASE OF TEMPERATURE

Let it be assumed that the temperature has decreased the same amount all through the depth of the pavement. What, then, is the effect upon the pavement?

In order to explain the analysis which follows it will be desirable to describe in general terms the behavior of the panel or slab of concrete when its temperature decreases uniformly. The immediate result is that there is a tendency for the slab to contract both longitudinally and transversely. The contraction is resisted by friction between the slab and the subgrade and as a result tensile stresses, both longitudinal and transverse, are set up in the concrete. These stresses tend to elongate the slab in the two directions in which they act and to shorten it in the direction at right angles to their line of action. The longitudinal tensile stress induces a longitudinal elongation and a transverse shortening. This transverse shortening bears to the longitudinal elongation the relation defined by Poisson's ratio, μ . The transverse tensile stress induces a transverse elongation and a longitudinal shortening. The ratio of this shortening, per unit of length, to the elongation, per unit of length, is Poisson's ratio, μ .

Consider, then, the action which takes place in the longitudinal direction: The slab contracts as a result of the decreased temperature; the contraction is resisted by the friction offered by the subgrade, and the longitudinal tensile stress thus set up causes an elongation which diminishes the amount of contraction that would otherwise occur; but the transverse tensile stress, which is set up in the same way by friction in the transverse direction, again increases the longitudinal contraction; so that the net result is a combination of three tendencies.

The elongations per unit of length resulting are expressed by the following equations:

$$\epsilon_x = \frac{1}{E}(\sigma_x - \mu\sigma_y) - \epsilon_t, \quad \epsilon_y = \frac{1}{E}(\sigma_y - \mu\sigma_x) - \epsilon_t \quad (1)$$

in which x, y = horizontal rectangular coordinates

σ_x, σ_y = normal tensile stresses in the directions of x or y , respectively,

ϵ_x, ϵ_y = elongations per unit of length in the directions of x or y , respectively,

E = modulus of elasticity of concrete,

μ = Poisson's ratio for the concrete,

ϵ_t = coefficient of temperature expansion,

t = decrease of temperature.

At a corner of an unloaded panel of pavement, obviously, there can be no stresses σ_x and σ_y . At a small distance from the corner, stresses may develop if the friction offered by the subgrade is considerable. In that case the loads are the least likely to produce a corner break, because the existence of considerable subgrade friction would imply a strong subgrade support. The uniform drop of temperature, therefore, is not likely to be important as a contributory cause of a corner break.

Consider next the central area of a large panel. Assume that the friction is sufficient to prevent the slab from contracting in either direction, i. e., to make $\epsilon_x=0$ and $\epsilon_y=0$. Then by solving the two equations 1, one finds

$$\sigma_x = \sigma_y = \frac{E\epsilon_t}{1-\mu} \quad (2)$$

Consider finally an edge of the panel, parallel to the axis of x . Consider a point of this edge at an appreciable distance from any corner, and assume the friction to be sufficient to make $\epsilon_x=0$. Obviously there can be no stress perpendicular to the edge at the edge. With $\sigma_y=0$ one finds by equations 1, at the edge,

$$\sigma_x = E\epsilon_t \quad (3)$$

The stresses given by equations 2 and 3 may have to be added to those caused by the loads.¹

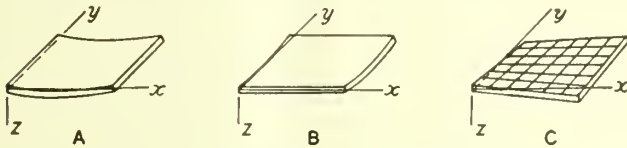


FIG. 1.—DEFORMATIONS OF ELEMENT OF SLAB

In a numerical example assume $E=3,000,000$ pounds per square inch, $\mu=0.15$, $\epsilon_t=0.0000060$ per degree Fahrenheit, and $t=50^\circ$ F. Then equations 2 and 3 give $\sigma_x=\sigma_y=1,059$ pounds per square inch in the central area, and $\sigma_x=900$ pounds per square inch at the edge. One may note that a shrinkage of 0.0003 inch per inch, an entirely possible value, would produce the same stresses, which, it will be noted, are those that would be developed if the concrete were capable of withstanding without shrinkage or cracking a drop in temperature of 50° F.

The possibility exists both in the case of low temperature in the cold season and in the case of shrinkage that a plastic flow of the concrete may relieve these stresses appreciably. Otherwise the large stresses computed can be relieved only by the presence of joints or the forming of cracks, combined with a sufficient amount of sliding of the pavement on the subgrade.²

CURLING STRESSES AND DEFLECTIONS

Assume next that the temperature remains normal at the middle plane of the pavement, but is t degrees lower at the top of the pavement than at the bottom.

Assume a uniform temperature gradient through the thickness, h , of the slab. Assume that the reactions of the subgrade are vertical only, and equal to kz per unit of area, where z is the deflection (positive downward) and k is a constant "modulus of subgrade reaction."³

Under the assumed temperature condition—i. e., the temperature lower at the top than at the bottom—the slab will tend to curl upward at the edge, and its state of flexure may be resolved into three component parts as follows: Bending in the x -direction (fig. 1, A), bending in the y -direction (fig. 1, B), and torsion or twisting in the xy -directions (fig. 1, C). The amounts of the deformations are measured in Figures 1, A and

B, by the curvatures $-\frac{\partial^2 z}{\partial x^2}$ and $-\frac{\partial^2 z}{\partial y^2}$ (as in beams), and in Figure 1, C, by the rate of change of slope; that is, by $-\frac{\partial^2 z}{\partial x \partial y}$.

If now, we denote by M_x and M_y the bending moments in the directions of x and y , respectively, per unit of width of cross section, and by M_z the twisting moment in the directions of x and y per unit of width of cross section,⁴ then the curvatures of the elastic surface (the bent middle plane) in the directions of x and y , respectively, will be:

$$\left. \begin{aligned} -\frac{\partial^2 z}{\partial x^2} &= \frac{12}{Eh^3}(M_x - \mu M_y) + \frac{\epsilon_t t}{h}, \\ -\frac{\partial^2 z}{\partial y^2} &= \frac{12}{Eh^3}(M_y - \mu M_x) + \frac{\epsilon_t t}{h}, \end{aligned} \right\} \quad (4)$$

and the twist in the directions of x and y will be:

$$-\frac{\partial^2 z}{\partial x \partial y} = \frac{12(1+\mu)}{Eh^3} M_z \quad (5)$$

The thickness, h , and the difference of temperature, t , will be assumed from here on to be the same at all points of the slab.

Center stress of a broad and long slab.—Consider first the central area of a long and broad unloaded panel. At a sufficient distance from the edges the deflections, z , must be zero, i. e., the curvatures of the slab must be zero. Then the equations just stated give

$$M_x = M_y = -\frac{Eh^2 \epsilon_t t}{12(1-\mu)}, \quad M_z = 0 \quad (6)$$

The corresponding tensile stress at the top is found by dividing $-M_x$ by the section modulus per unit of width, $\frac{h^2}{6}$; and this tensile stress, which is the same in all horizontal directions, is

$$\sigma_o = \frac{E\epsilon_t t}{2(1-\mu)} \quad (7)$$

By substitution, then, with $E=3,000,000$ pounds per square inch, $\mu=0.15$, and $\epsilon_t=0.0000060$ per degree Fahrenheit, as before, and $t=10^\circ$ F., one finds the

¹ WESTERGAARD, H. M. STRESSES IN CONCRETE PAVEMENTS COMPUTED BY THEORETICAL ANALYSIS. PUBLIC ROADS, April, 1926 [or COMPUTATION OF STRESSES IN CONCRETE ROADS. Proceedings of the fifth annual meeting of the Highway Research Board, December 3-4, 1925, Pt. 1, p. 90, 1926]. Tables 3 and 4 in this paper contain values of the stresses produced by a wheel load of 10,000 pounds in the central area and at the edge, respectively.

² For a discussion of the spacing of joints required to prevent cracking, see GOLDBECK, A. T., THE INTERRELATION OF LONGITUDINAL STEEL AND TRANSVERSE CRACKS IN CONCRETE ROADS, PUBLIC ROADS, vol. 6, No. 6, August, 1925.

³ A discussion of the assumptions involved here, especially in regard to the reactions of the subgrade, may be found in the paper entitled, "Stresses in concrete pavements computed by theoretical analysis," PUBLIC ROADS, April, 1926.

⁴ Concerning the character and action of these moments, see WESTERGAARD, H. M., and SLATER, W. A., MOMENTS AND STRESSES IN SLABS. Proc. Am. Concrete Inst. v. 17, 1921, p. 415, especially, p. 424, or, Nat. Research Council, Reprint and Circular Series, No. 32, especially p. 10.

stress at the top to be $\sigma_o = 106$ pounds per square inch.

This, it may be well to repeat, is the stress which would be caused by a difference of temperature of 10° in the central area of a broad and long slab, where, by reason of the distance from the edge, there is no deflection.

With $t = -10^\circ \text{ F.}$, that is, with the temperature 10° higher at the top than at the bottom, the same tensile stress will be found at the bottom of the pavement, and, in this case, may have to be added to the stress produced by wheel loads.⁵

Deflection and stress at points near the edge of a broad and long slab.—Consider next a slab which has an edge along the axis of x , and which extends infinitely far in the directions of positive and negative x and positive y . With the difference of temperature between top and bottom the same all over the slab, the deflection, z , is a function of y only, independent of x .

One finds, then, by eliminating M_x in equations 4:

$$M_y = \frac{Eh^3}{12(1-\mu^2)} \left(-\frac{d^2 z}{dy^2} - \frac{(1+\mu)\epsilon t}{h} \right) \text{-----} (8)$$

The reactions of the subgrade, kz , are the only external forces to be taken into consideration. The expression kz may be applied when z is negative (upward) provided that there is still contact between the slab and the subgrade. This contact may be maintained by the weight of the pavement or by this weight in conjunction with loads the effects of which are to be added afterwards to the effects of the variation of temperature.

The equilibrium of any small element of the slab requires, accordingly,

$$\frac{d^2 M_y}{dy^2} = kz \text{-----} (9)$$

By substituting M_y from equation 8 one finds:

$$\frac{Eh^3}{12(1-\mu^2)} \frac{d^4 z}{dy^4} + kz = 0 \text{-----} (10)$$

Here, as in other investigations of slabs on elastic subgrade, it is expedient to introduce the linear distance,

$$l = \sqrt[4]{\frac{Eh^3}{12(1-\mu^2)k}} \text{-----} (11)$$

which is called the "radius of relative stiffness." This distance is a measure of the stiffness of the slab relative to the stiffness of the subgrade. Table 1 gives values of the radius of relative stiffness for different values of h and k .⁶

TABLE 1.—Values of the radius of relative stiffness, l , for different values of the slab thickness, h , and of the modulus of subgrade reaction, k , computed from equation 11

[$E = 3,000,000$ pounds per square inch, $\mu = 0.15$]

Thickness of slab in inches h	Radius of relative stiffness, l , in inches		
	$k = 50 \text{ lb./in.}^3$	$k = 100 \text{ lb./in.}^3$	$k = 200 \text{ lb./in.}^3$
4-----	23.91	20.11	16.92
5-----	28.28	23.78	20.00
6-----	32.40	27.26	22.92
7-----	36.40	30.60	25.73
8-----	40.23	33.83	28.44
9-----	43.94	36.95	31.07
10-----	47.55	40.00	33.62
11-----	51.08	42.94	36.11
12-----	54.52	45.84	38.56

With l introduced, equation (10) becomes:

$$l^4 \frac{d^4 z}{dy^4} + z = 0 \text{-----} (12)$$

In the present case the functions z and $\frac{dz}{dy}$ must converge toward zero when y increases indefinitely. Furthermore, at the edge, $y = 0$, the bending moment, M_y , and the vertical shear, $\frac{dM_y}{dy}$, must also be zero. The following solution is correct because it satisfies these end conditions as well as equation 12:

$$z = -z_o \sqrt{2} \cos \left(\frac{y}{l\sqrt{2}} + \frac{\pi}{4} \right) e^{-\frac{y}{l\sqrt{2}}} \text{-----} (13)$$

where e is the base of the Napierian system of logarithms⁵ and

$$z_o = \frac{(1+\mu)\epsilon t}{h} l^2 = \epsilon t \sqrt{\frac{(1+\mu)Eh}{12(1-\mu)k}} \text{-----} (14)$$

is the reversed (upward) deflection at the edge, $y = 0$.

By substituting these expressions in equation (8) one finds the bending moment M_y . By dividing $-M_y$ by the section modulus per unit of width, $\frac{h^2}{6}$, the result is the tensile stress at the top of the slab in the direction of y ,

$$\sigma_y = \sigma_o \left(1 - \sqrt{2} \sin \left(\frac{y}{l\sqrt{2}} + \frac{\pi}{4} \right) e^{-\frac{y}{l\sqrt{2}}} \right) \text{-----} (15)$$

where

$$\sigma_o = \frac{E\epsilon t}{2(1-\mu)} \text{-----} (16)$$

(same as equation 7)

By solving the first of equations 4 for M_x with $\frac{d^2 z}{dx^2} = 0$, and dividing $-M_x$ by the section modulus, $\frac{h^2}{6}$

⁵ See Table 3 in the paper, "Stresses in concrete pavements computed by theoretical analysis."

⁶ Table 1 is the same as Table 1 in the paper, "Stresses in concrete pavements computed by theoretical analysis."

one finds the tensile stress at the top, in the direction of x ,

$$\sigma_x = \frac{E\epsilon_t t}{2} + \mu\sigma_y = \sigma_o + \mu(\sigma_y - \sigma_o), \text{-----} (16)$$

or, by substituting σ_y from equation 15,

$$\sigma_x = \sigma_o \left(1 - \mu\sqrt{2} \sin \left(\frac{y}{l\sqrt{2}} + \frac{\pi}{4} \right) e^{-\frac{y}{l\sqrt{2}}} \right) \text{----} (17)$$

These expressions for z , σ_y and σ_x (equations 13, 15, and 17) all apply to the condition of a slab of infinite extent in the directions of x and y . The function, z , is the deflection in the transverse direction occurring at

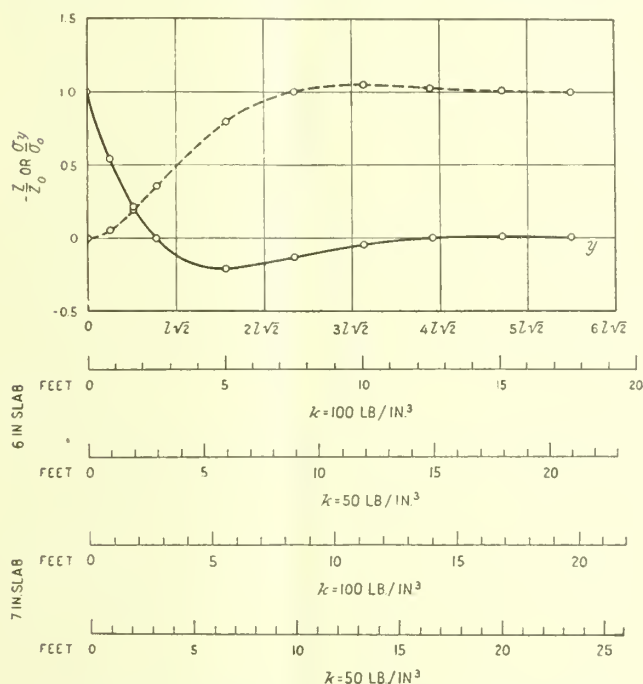


FIG. 2.—THE CURLING OF A PAVEMENT WHEN THE TEMPERATURE IS NORMAL AT THE MIDDLE PLANE, AND t DEGREES LOWER AT THE TOP THAN AT THE BOTTOM. THE PAVEMENT, INFINITELY WIDE IN THE DIRECTION OF y , COVERS THE WHOLE POSITIVE SIDE OF THE AXIS OF x . THE HORIZONTAL DISTANCES, y , ARE MEASURED FROM THE EDGE. THE ORDINATES OF THE FULL-LINE CURVE REPRESENT RELATIVE VALUES OF THE DEFLECTIONS, z , AND THOSE OF THE DOTTED CURVE REPRESENT RELATIVE VALUES OF THE TENSILE STRESSES, σ_y , AT THE TOP OF THE PAVEMENT IN THE DIRECTION PERPENDICULAR TO THE EDGES. SCALES FOR y ARE GIVEN IN TERMS OF l FOR GENERAL APPLICATION, AND NUMERICALLY IN FEET FOR SLAB THICKNESSES OF 6 AND 7 INCHES AND VALUES OF k EQUAL TO 50 AND 100 POUNDS PER CUBIC INCH FOR EACH SLAB THICKNESS

any distance, y , from the edge of the slab and it is expressed in equation (13) in terms of z_o , the deflection of the edge. The stresses σ_x and σ_y are principal stresses, one the greatest, the other the smallest at the particular point, and both are expressed (equations 15 and 17) in terms of the stress, σ_o , corresponding to points in the central area of the slab at which, by reason of distance from the edge there is no deflection. There are no shearing stresses in the directions of x and y .

Although this case is not similar to that of any ordinary pavement condition, since pavement widths

are relatively narrow, the relative values of z in terms of z_o and of σ_y in terms of σ_o are shown by the full and dotted curves, respectively, in Figure 2. The general scale, giving distances from the edge in terms of l , is converted into feet for thicknesses of 6 and 7 inches and for values of k equal to 100 and 50 pounds per cubic inch, in accordance with the values given in Table 2. The same table gives also the corresponding values of the edge deflection, z_o , the stress σ_o being for all thicknesses and values of k the same and equal to 106 pounds per square inch (see example under equation 7). By reference to the scales in feet it will be observed that the stress of 106 pounds per square inch and zero deflection obtain at all points more distant than 18 feet from the edge in a 6-inch slab with a modulus of subgrade reaction equal to 100 and varying from this distance to 24 feet for the 7-inch slab with a subgrade modulus of 50.

TABLE 2.—Examples of values of the reversed deflection, z_o , at the edge in the case of Figure 2, computed from equation 14, for $E=3,000,000$ pounds per square inch, $\mu=0.15$

[$\epsilon_t=0.000060$ per degree Fahrenheit, $t=10^\circ$ F. for typical values of the thickness of the slab, h , and the modulus of subgrade reaction, k . Corresponding value of the stress σ_o according to equation (7). $\sigma_o=106$ pounds per square inch. The values of l are taken from Table 1]

h inches	k lb./in. ³	l inches	z_o inches
6	100	27.26	0.0085
6	50	32.40	.0121
7	100	30.60	.0092
7	50	36.40	.0131

The case of a long slab of relatively narrow width.—The next case to consider is that of an infinitely long strip or slab of finite width b . Let the center line be along the axis of x . Then the edges have the equations, $y=\pm \frac{b}{2}$. This case corresponds approximately to that of a newly constructed pavement not yet cracked transversely, in which the unjointed length is great in comparison with the width.

Assume again the difference of temperature between the top and the bottom of the slab to be constant and equal to t . Then, as in the previous case, the deflection, z , will be a function of y only, independent of x . The flexure will be governed again by equations 8, 9, 10, 12, and 16. The end conditions in this case are: $M_y=0$, $\frac{dM_y}{dx}=0$, at the edges $y=\pm \frac{b}{2}$. The following solution is correct because it satisfies these end conditions and the differential equation 12:⁷

$$z = -z_o \frac{2 \cos \lambda \cos h\lambda}{\sin 2\lambda + \sin h2\lambda} \left[(-\tan \lambda + \tan h\lambda) \cos \frac{y}{l\sqrt{2}} \cos h \frac{y}{l\sqrt{2}} + (\tan \lambda + \tan h\lambda) \sin \frac{y}{l\sqrt{2}} \sin h \frac{y}{l\sqrt{2}} \right] \text{-----} (18)$$

where

$$\lambda = \frac{b}{l\sqrt{8}} \text{-----} (19)$$

and, as before,

$$z_o = \frac{(1+\mu)\epsilon_t t}{h} l^2 = \epsilon_t l \sqrt{\frac{(1+\mu)Eh}{12(1-\mu)k}} \text{ (same as equation 14)}$$

⁷ Equations 13 and 18 and corresponding expressions for M_x and M_y were stated in a paper by the writer, OM BEREKNING AF PLADER PAA ELASTISK UNDERLAG MED SAERLIGT HENBLIK PAA SPØRGSMÅLET OM SPAENDINGER I BETONVEJE. Ingeniøren, v. 32, p. 513, 1923, Copenhagen. See p. 524.

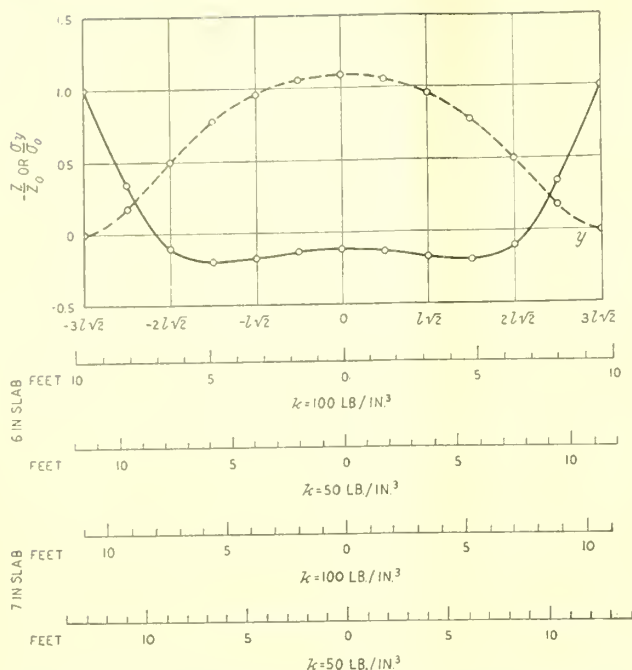


FIG. 3.—THE CURLING OF A PAVEMENT OF WIDTH $b = 6l\sqrt{2}$ WHEN THE TEMPERATURE IS NORMAL AT THE MIDDLE PLANE, AND t DEGREES LOWER AT THE TOP THAN AT THE BOTTOM. THE HORIZONTAL DISTANCES, y , ARE MEASURED FROM THE CENTER LINE OF A STRIP OF PAVEMENT INCLOSED BETWEEN THE LINES $y = \pm \frac{b}{2}$. THE ORDINATES OF THE FULL-LINE CURVE REPRESENT RELATIVE VALUES OF THE DEFLECTIONS, z , AND THOSE OF THE DOTTED CURVE REPRESENT RELATIVE VALUES OF THE TENSILE STRESSES, σ_y , AT THE TOP OF THE PAVEMENT IN THE DIRECTION PERPENDICULAR TO THE EDGES. SCALES FOR y ARE GIVEN IN TERMS OF l FOR GENERAL APPLICATION, AND NUMERICALLY IN FEET FOR SLAB THICKNESSES OF 6 AND 7 INCHES AND VALUES OF k EQUAL TO 50 AND 100 POUNDS PER CUBIC INCH FOR EACH SLAB THICKNESS

By the process applied in the preceding case one finds the corresponding principal stresses at the top of the slab:

$$\sigma_y = \sigma_o \left[1 - \frac{2 \cos \lambda \cos h\lambda}{\sinh 2\lambda + \sin h2\lambda} \left((\tan \lambda + \tan h\lambda) \cos \frac{y}{l\sqrt{2}} \cos h \frac{y}{l\sqrt{2}} + (\tan \lambda - \tan h\lambda) \sin \frac{y}{l\sqrt{2}} \sin h \frac{y}{l\sqrt{2}} \right) \right] \quad (20)$$

$$\sigma_x = \sigma_o + \mu (\sigma_y - \sigma_o), \quad (21)$$

where, as before,

$$\sigma_o = \frac{E\epsilon_t t}{2(1-\mu)} \quad (\text{same as equation 7})$$

Numerical computations by equations 18 and 20 become a simple matter by use of a book of tables published by Hayashi in 1921.⁸ These tables give the values of the hyperbolic and natural sines and cosines in a convenient form, and they have been used in making the computations given in Tables 3 and 4 and graphically in Figures 2, 3, 4, and 5.

Table 3 and Figures 3 and 4 show the relative deflections and stresses at different distances from the center line of a pavement of two different widths, one half as great as the other. Deflections are expressed as functions of the edge deflection of a slab of infinite width, and the stresses are expressed in terms of σ_o , the stress in

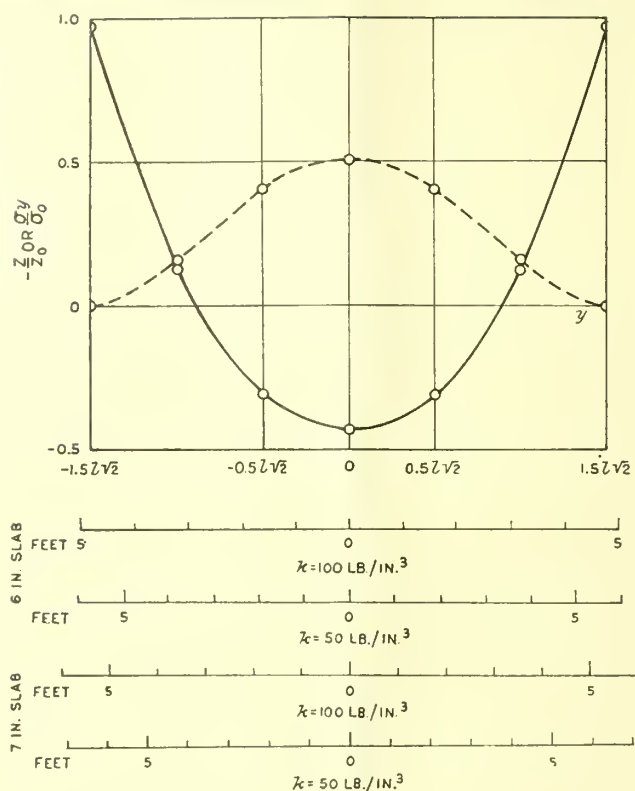


FIG. 4.—THE CURLING OF A PAVEMENT OF WIDTH $b = 3l\sqrt{2}$ (ONE-HALF THE WIDTH OF THE PAVEMENT REPRESENTED BY FIGURE 3), WHEN THE TEMPERATURE IS NORMAL AT THE MIDDLE PLANE, AND t DEGREES LOWER AT THE TOP THAN AT THE BOTTOM. THE HORIZONTAL DISTANCES, y , ARE MEASURED FROM THE CENTER LINE OF THE STRIP OF PAVEMENT INCLOSED BETWEEN THE LINES $y = \pm \frac{b}{2}$. THE ORDINATES OF THE FULL-LINE CURVE REPRESENT RELATIVE VALUES OF THE DEFLECTIONS, z , AND THOSE OF THE DOTTED CURVE REPRESENT RELATIVE VALUES OF THE TENSILE STRESSES, σ_y , AT THE TOP OF THE PAVEMENT IN THE DIRECTION PERPENDICULAR TO THE EDGES. SCALES FOR y ARE GIVEN IN TERMS OF l FOR GENERAL APPLICATION, AND NUMERICALLY IN FEET FOR SLAB THICKNESSES OF 6 AND 7 INCHES AND VALUES OF k EQUAL TO 50 AND 100 POUNDS PER CUBIC INCH FOR EACH SLAB THICKNESS

the central portion of such an infinite slab. By comparison of the two figures and the columns of the table corresponding to the two widths one sees clearly the effect upon deflections and stresses of cutting the continuous width of a pavement slab in half by the introduction of a center joint. By so doing, for example, it appears that the edge deflections are not appreciably altered, but the maximum stress is reduced by more than half.

According to the numerical scales at the bottom of Figures 3 and 4, the diagram (fig. 3) may represent, for example, a pavement 20 feet wide without center joint and still uncracked. The maximum stress, σ_y , at the center (see also Table 3) is $1.084\sigma_o$. If this pavement cracks along the center line, the left half of it will be represented by the diagram (fig. 4), and, as will be seen, the maximum value of σ_y becomes $0.508\sigma_o$. With $\mu = 0.15$ the corresponding stresses, σ_x , in the direction of the road become, according to equation 21:

In the case represented by Figure 3,

$$\sigma_x = \sigma_o (1 + 0.15 \times 0.084) = 1.013\sigma_o$$

⁸ HAYASHI, K. KUNSTSTÄLLE TAFELN DER KREIS-UND HYPERBELFUNKTIONEN SOWIE DER FUNKTIONEN e^x UND e^{-x} MIT DEN NATÜRLICHEN ZAHLEN ALS ARGUMENT. Berlin und Leipzig, 1921. (Walter de Gruyter & Co.)

and in the case represented by Figure 4,

$$\sigma_x = \sigma_o (1 - 0.15 \times 0.492) = 0.926 \sigma_o.$$

That is, in order that the stress σ_x shall be relieved in a manner comparable to that of σ_y , transverse cracks or joints are required. At the edge, without transverse cracks or joints, the stresses are the same before and after the longitudinal crack has formed, being, respectively,

$$\sigma_y = 0 \text{ and } \sigma_x = 0.85 \sigma_o.$$

TABLE 3.—Two examples of curling of a strip of pavement of width b , when the temperature at the top of the pavement is t degrees lower than at the bottom

Relative values of the deflections, z , and the stresses, σ_y , across the strip, at different distances, y , from the center line, from equations 18 and 20. Graphical representation in Figures 3 and 4. Examples of numerical values of z_o and σ_o are given in Table 2]

$\frac{y}{l\sqrt{2}}$	$b=6l\sqrt{2}$		$b=3l\sqrt{2}$	
	$\frac{z}{z_o}$	$\frac{\sigma_y}{\sigma_o}$	$\frac{z}{z_o}$	$\frac{\sigma_y}{\sigma_o}$
0	0.1126	1.084	0.4320	0.508
0.5	.1326	1.056	.3049	.405
1	.1773	.959	-.1260	.163
1.5	.1980	.775	-.9720	.000
2	.1021	.493	-----	-----
2.5	-.3476	.178	-----	-----
3	-1.003	.000	-----	-----

TABLE 4.—Curling of a strip of pavement of width b , when the temperature at the top of the pavement is t degrees lower than at the bottom

[Relative values of the deflections, z_e , at the edge, and of the stresses, σ_e , across the center line, for different values of b , computed from equations 18 with $y=\frac{b}{2}$, and 20 with $y=0$. Graphical representation in Figure 5. Examples of numerical values of z_o and σ_o are given in Table 2]

$\frac{b}{l\sqrt{8}} (= \lambda)$	$\frac{b}{l}$	$\frac{z_e}{z_o}$	$\frac{\sigma_e}{\sigma_o}$
∞	∞	-1.000	1.000
4	11.31	-0.999	1.052
3	8.49	-1.003	1.084
2.5	7.07	-1.026	1.032
2.365	6.69	-1.036	1.000
2	5.66	-1.057	0.856
1.571	4.44	-1.000	-----
1.5	4.24	-0.972	0.508
1	2.828	-0.599	0.148
0.5	1.414	-0.165	0.010

TABLE 5.—Example of stresses, in pounds per square inch, occurring in a 20-foot pavement without transverse cracks or joints, before and after a longitudinal crack has formed along the center line

[The temperature at the top of the pavement is assumed to be 10° F. lower than at the bottom. $E=3,000,000$ pounds per square inch; $\mu=0.15$; $\epsilon_t=0.0000060$ per degree Fahrenheit; $\sigma_o=105$ pounds per square inch (compare Table 2)]

	Before cracking longitudinally		After cracking longitudinally	
	Center of 20-foot strip	Edge	Center of 10-foot strip	Edge
Transverse stress, σ_y (pounds per square inch) -----	115	0	54	0
Longitudinal stress, σ_x (pounds per square inch) -----	107	90	97	90

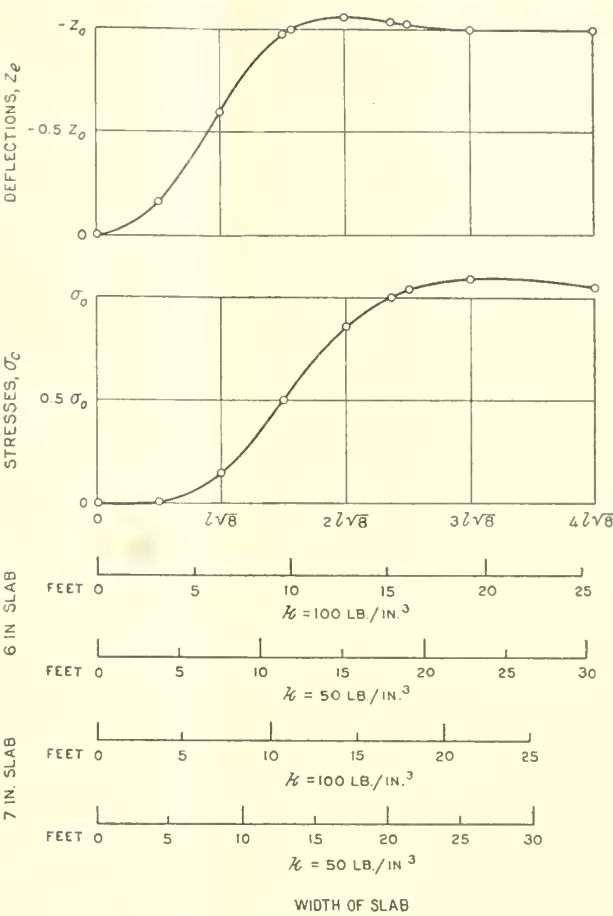


FIG. 5.—Curling of a strip of pavement of width b , when the temperature is normal at the middle plane, and t degrees lower at the top than at the bottom, giving values of the deflections, z_e , at the edge, and the stresses, σ_e , across the center line, for different values of b . The scales for b at the bottom of the figure apply for the particular values of h and k stated in Table 2

Table 5 shows numerical values obtained by this procedure on the basis of the values assumed in connection with Table 2. Stresses like those in Table 5 may have to be combined with the stresses resulting from a superimposed uniform drop of temperature (drop of temperature in the middle plane of the slab), with forces of friction active between the slab and the subgrade.

By assuming $t = -10^\circ$ F. instead of $+10$ degrees—that is, by assuming the top of the pavement to be 10 degrees warmer than the bottom, the tensile stresses given in Table 5 will still exist, but will be at the bottom of the pavement instead of at the top. These tensions at the bottom are significant in that they may have to be combined with the tensions due to wheel loads, both in the central area and at the edge.⁹

Edge deflections and center stresses tabulated for purposes of practical design.—In Table 4 and Figure 5 are given the relative edge deflections and center stresses in slabs of various widths corresponding to the several

⁹ Values of these stresses due to wheel loads are given in Tables 3 and 4 in the paper "Stresses in concrete pavements computed by theoretical analysis."

values of b . Here again the deflections and stresses are given in terms of the edge deflection and interior stress, respectively, of a slab of infinite width, and the values given for both are maxima for the several widths represented.

Since these factors, i. e., the edge deflection and the center stress for slabs of finite width, are of greatest practical importance for purposes of design, Tables 6 and 7 have been compiled to show the numerical values of the two factors for a range of thickness and width of slab embracing the common dimensions and for three values of the modulus of subgrade reaction, k . The values given in these tables are based on a difference of temperature of 10° F. between the top and bottom of the slab. Values corresponding to any other difference of temperature may be obtained by multiplying the tabulated values by $\frac{t}{10}$, where t represents the difference of temperature assumed for purposes of design.

TABLE 6.—Edge deflections of concrete slabs of infinite length¹ and various widths and thicknesses, corresponding to three values of the modulus of subgrade reaction, when the difference of temperature between the top and bottom of the slab is 10° F.

[$E=3,000,000$ pounds per square inch, $\mu=0.15$, $\epsilon_1=0.0000060$]

Modulus of subgrade reaction k	Thickness of slab in inches	Width of slab in feet							
		9	12	14	16	18	20	24	30
		Edge deflection in inches							
50-----	6	0.0092	0.0121	0.0127	0.0127	0.0125	0.0123	0.0121	0.0121
	7	.0085	.0121	.0133	.0138	.0138	.0136	.0132	.0130
	8	.0077	.0116	.0134	.0144	.0148	.0147	.0143	.0140
	10	.0062	.0104	.0128	.0146	.0158	.0164	.0164	.0158
100-----	6	.0079	.0090	.0090	.0088	.0086	.0086	.0086	.0086
	7	.0076	.0095	.0098	.0097	.0095	.0093	.0092	.0092
	8	.0071	.0096	.0103	.0104	.0103	.0101	.0099	.0099
	10	.0061	.0093	.0107	.0114	.0117	.0116	.0113	.0110
200-----	6	.0062	.0063	.0062	.0061	.0060	.0060	.0060	.0060
	7	.0063	.0069	.0068	.0066	.0066	.0065	.0065	.0065
	8	.0062	.0073	.0074	.0072	.0071	.0070	.0070	.0070
	10	.0056	.0076	.0082	.0082	.0081	.0080	.0078	.0078

¹ For practical purposes the values given in the table apply also to slabs the length of which with respect to the width is relatively great.

TABLE 7.—Stresses across the center line of concrete slabs of infinite length¹ and various widths and thicknesses, corresponding to three values of the modulus of subgrade reaction, when the difference of temperature between the top and bottom of the slab is 10° F.

[$E=3,000,000$ pounds per square inch, $\mu=0.15$, $\epsilon_1=0.0000060$]

Modulus of subgrade reaction k	Thickness of slab in inches	Width of slab in feet							
		9	12	14	16	18	20	24	30
		Center stress in pounds per square inch							
50-----	6	27	60	80	96	106	112	115	112
	7	18	45	65	82	96	105	114	114
	8	13	34	52	70	85	96	110	115
	10	10	20	33	48	63	77	98	112
100-----	6	45	83	99	109	114	115	113	104
	7	33	68	87	101	109	113	115	110
	8	24	54	75	91	102	110	115	113
	10	13	35	53	70	85	97	110	115
200-----	6	68	101	111	115	115	113	109	106
	7	53	90	104	112	115	115	112	107
	8	40	77	95	107	112	115	114	109
	10	24	55	75	92	103	110	115	113

¹ For practical purposes the values given in the table apply also to slabs the length of which with respect to the width is relatively great.

Stresses in a rectangular panel.—It remains to discuss the case of a rectangular panel. Let the edges have

the equations $x = \pm \frac{B}{2}$ and $y = \pm \frac{b}{2}$. Let $z = f(y)$ represent the solution for $B = \infty$ and b finite, and let $z = F(x)$ represent the analogous solution for B finite and $b = \infty$. Assume for the time being that Poisson's ratio is zero. Then, with the finite values of both B and b , the following solution will satisfy all the conditions:

$$z = f(y) + F(x). \quad (22)$$

With this solution the stresses σ_y may be found as if B were infinite, and the stresses σ_x as if b were infinite. It is a fact, however, that Poisson's ratio, μ , is not equal to zero. With $\mu=0.15$ the influence of this ratio is exemplified in Table 5, where the stresses σ_x vary from 90 to 107 pounds per square inch. If Poisson's ratio were zero, these stresses would all be 90 pounds per square inch. The variation is a matter of 18 per cent. If errors of this magnitude can be tolerated in a first approximation, then the solution indicated by equation 22 may be accepted as a first approximation. It may be noted that the radius of relative stiffness, l (see equation 11) is affected only slightly by a variation of Poisson's ratio, and the values of l given in Table 1, therefore, may still be used.

This consideration will be applied for the purpose of estimating the influence of curling due to a difference of temperature upon the corner break. The stresses due to the variation of temperature must be combined, in this case, with the stresses due to a wheel load acting close to the corner. Assume that the slab is 7 inches thick and that the resultant of a wheel load equal to 5,000 pounds is applied 4 inches from each of two edges which form the rectangular corner. Assume that the modulus of subgrade reaction is $k=100$ pounds per cubic inch. Then Table 1 gives $l=30.60$ inches. According to a previous paper¹⁰ the load produces in this case a tensile stress at the top equal to $\sigma_c = 0.5 \times 390 = 195$ pounds per square inch. This stress occurs in a section which intersects the edges at a distance from the corner approximately equal to

$$x_1 \sqrt{2} = 2 \sqrt{2} \sqrt{4 \sqrt{2} \times 30.60} = 37.2 \text{ inches} = 0.860 l \sqrt{2}.$$

Both Figure 2 and Figure 3 indicate at this distance from the left of the diagram a temperature stress $\sigma = 0.40\sigma_o$. Figure 4 gives $\sigma = 0.34\sigma_o$. Since the maximum temperature stress occurs at a greater distance from the corner, one may obtain the maximum combined stress due to temperature and load by combining the stress due to load, $\sigma_c = 195$ pounds per square inch, with a temperature stress somewhat larger than $0.40\sigma_o$. The value $\sigma = 0.5\sigma_o$ may be considered to be a reasonable estimate. With $t=10$ degrees Fahrenheit, and the other numerical values as in the previous examples, except that $\mu=0$, one finds $\sigma_o = 90$ pounds per square inch, that is, $\sigma = 45$ pounds per square inch. Since the stress σ_c belongs in a section making an angle of 45 degrees with the edge, and the stress σ in a section perpendicular to the edge, the combined maximum stress will be, approximately, $\sigma_c + \frac{1}{2}\sigma = 195 + \frac{1}{2} \times 45 = 218$ pounds per square inch.

¹⁰ "Stresses in concrete pavements computed by theoretical analysis," Table 2, and equation 4.

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Report of the Chief of the Bureau of Public Roads, 1925.

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- Vol. 11, No. 10, D-15. Tests of a Large-Sized Reinforced-Concrete Slab Subjected to Eccentric Concentrated Loads.

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UNITED STATES DEPARTMENT OF AGRICULTURE
BUREAU OF PUBLIC ROADS
STATUS OF FEDERAL AID HIGHWAY CONSTRUCTION

AS OF

APRIL 30, 1927

STATES	FISCAL YEARS 1917-1926						FISCAL YEAR 1927						BALANCE OF FEDERAL AID FUND AVAILABLE FOR NEW PROJECTS	STATES
	PROJECTS COMPLETED PRIOR TO JULY 1, 1926			PROJECTS COMPLETED SINCE JUNE 30, 1926			* PROJECTS UNDER CONSTRUCTION							
	TOTAL COST	FEDERAL AID	MILES	TOTAL COST	FEDERAL AID	MILES	ESTIMATED COST	FEDERAL AID ALLOTTED	MILES	ESTIMATED COST	FEDERAL AID ALLOTTED	MILES		
Alabama	\$ 16,226,411.34	\$ 7,725,985.09	1258.3	\$ 1,834,960.34	\$ 889,114.85	101.9	\$ 6,566,145.16	\$ 3,127,211.59	381.3	\$ 250,535.48	\$ 136,675.57	14.3	Alabama	
Arizona	5,653,772.35	2,663,772.35	729.6	741,370.79	508,779.34	48.6	1,166,448.59	791,164.87	73.0	244,878.65	176,216.27	11.7	Arizona	
Arkansas	18,354,544.50	7,666,639.35	1323.0	3,825,333.39	1,788,703.69	227.7	2,446,897.77	1,179,389.37	199.5	1,004,351.69	501,815.84	39.3	Arkansas	
California	27,142,596.90	13,003,592.30	1058.0	7,751,753.63	3,834,300.45	240.6	7,355,382.51	3,419,674.51	152.9	89,175.17	37,146.53	2.1	California	
Colorado	13,905,304.64	7,127,288.18	745.0	1,500,384.22	768,903.83	72.4	6,348,324.16	3,075,282.41	278.3	166,779.97	104,700.27	12.1	Colorado	
Connecticut	5,414,567.19	2,100,585.80	117.1	684,812.28	245,719.74	13.7	5,693,250.98	1,575,579.88	70.4	189,283.32	94,641.66	0.3	Connecticut	
Delaware	4,918,562.29	1,781,655.60	124.3	1,039,483.07	452,067.18	28.0	831,515.88	361,826.40	31.7	271,652.15	63,316.77	8.4	Delaware	
Florida	3,826,630.56	1,654,432.86	132.9	3,544,176.05	1,803,550.28	112.1	8,676,845.85	3,777,276.07	168.3	1,190,401.52	398,091.78	24.2	Florida	
Georgia	24,791,206.97	11,654,237.86	1794.0	5,955,294.06	2,712,305.78	307.6	9,421,767.30	4,546,622.72	398.3	462,600.34	217,323.56	8.3	Georgia	
Idaho	44,116,611.86	20,619,995.74	1377.7	2,164,317.31	1,193,414.46	110.8	2,109,030.97	1,324,331.38	162.2	682,439.37	403,115.31	40.1	Idaho	
Illinois	18,943,425.67	8,172,125.19	534.3	4,208,434.39	2,055,293.14	143.4	8,475,356.55	4,039,284.18	304.4	6,063,197.47	3,034,099.60	211.5	Illinois	
Indiana	29,062,375.40	11,826,202.10	2114.2	4,811,414.00	2,254,019.86	331.0	11,697,891.73	5,346,201.12	536.9	4,471,438.94	1,814,732.08	110.5	Indiana	
Iowa	32,626,601.64	12,590,489.25	1160.6	3,941,804.35	1,847,691.56	292.3	12,948,000.49	5,122,199.09	649.5	1,998,065.33	751,823.22	127.8	Iowa	
Kansas	20,737,706.10	8,482,082.25	758.2	2,245,566.73	915,591.55	100.1	9,235,887.89	4,356,478.31	413.2	752,262.54	355,473.56	48.3	Kansas	
Kentucky	13,630,592.68	6,144,735.99	1055.9	1,911,816.79	881,581.86	111.0	4,234,832.21	1,993,413.53	149.7	26,200.66	13,100.33	3.0	Kentucky	
Louisiana	8,747,552.78	4,192,507.39	303.6	1,817,247.30	665,945.28	54.0	2,231,038.63	821,344.43	63.4	87,912.01	43,956.00	5.6	Louisiana	
Maine	10,324,943.10	5,112,391.22	423.3	865,260.83	411,947.05	54.5	814,475.47	373,185.64	33.9				Maine	
Maryland	18,353,757.71	6,657,690.62	374.5	1,768,155.87	604,492.53	24.9	5,124,690.48	1,367,469.57	83.2	887,123.10	193,770.00	12.9	Maryland	
Massachusetts	25,997,240.78	11,827,052.30	963.0	3,291,881.72	1,451,890.07	84.7	13,544,573.62	6,093,979.44	383.7	1,605,066.00	760,705.00	40.6	Massachusetts	
Michigan	37,170,986.95	15,696,116.56	3181.8	7,928,662.52	3,460,029.11	461.6	4,398,005.64	1,306,198.90	268.0	2,985,440.85	799,700.00	119.2	Michigan	
Minnesota	15,146,088.92	7,414,534.10	1129.0	2,299,831.37	1,146,635.10	138.0	6,924,252.88	3,370,848.33	372.0	1,186,182.97	575,034.21	65.2	Minnesota	
Mississippi	28,989,168.92	13,736,014.85	1543.2	11,276,538.34	4,977,652.55	344.1	10,775,569.47	4,319,947.16	286.4	1,754,125.97	791,595.43	84.3	Mississippi	
Missouri	11,400,381.81	6,333,465.89	1054.9	1,454,011.91	953,832.80	96.7	1,549,479.24	1,333,488.24	122.2	2,100,527.65	695,737.23	145.5	Missouri	
Montana	11,533,401.62	5,474,202.52	1768.3	4,243,144.09	2,085,953.65	428.2	10,963,610.88	5,404,351.04	1167.8	2,422,195.43	1,194,321.26	176.5	Montana	
Nebraska	7,559,135.51	5,130,934.55	538.8	2,731,465.04	2,351,683.67	300.4	1,639,091.96	1,418,223.54	197.1	1,418,223.54	1,418,223.54	197.1	Nebraska	
Nevada	4,992,659.60	2,371,450.07	237.6	846,458.09	386,537.45	26.4	647,214.00	303,802.51	18.7	201,149.13	95,178.55	7.4	Nevada	
New Hampshire	16,346,301.01	5,099,342.21	290.3	5,981,939.07	2,397,022.27	26.0	4,725,000.24	970,846.60	59.8	2,346,992.79	598,976.96	42.8	New Hampshire	
New Jersey	12,404,337.77	7,339,657.38	1427.0	826,192.76	501,532.01	71.4	3,443,308.01	2,684,953.75	240.6	7,917,333.00	1,762,327.50	98.9	New Jersey	
New Mexico	43,224,279.79	17,111,957.19	1197.0	9,680,900.60	3,355,217.96	216.8	35,451,253.00	9,080,878.95	565.2				New Mexico	
New York	27,009,419.47	11,177,371.94	1257.9	7,793,014.85	3,135,240.32	169.3	2,673,142.76	1,267,381.42	79.3	1,163,231.30	517,169.75	34.3	New York	
North Carolina	12,313,311.40	6,031,869.78	2193.1	3,659,709.95	1,694,463.60	95.3	4,392,469.56	2,426,064.74	642.5	2,113,123.00	1,056,417.74	264.8	North Carolina	
North Dakota	47,699,532.90	17,371,787.03	1324.1	4,221,669.59	1,993,589.73	150.5	11,343,652.56	4,369,902.25	329.8	653,696.16	219,093.22	12.8	North Dakota	
Ohio	28,241,950.33	13,159,999.15	1178.9	1,886,210.72	858,132.39	73.8	3,259,630.75	1,366,982.43	203.9	1,977,435.69	898,603.25	119.1	Ohio	
Oklahoma	17,027,878.42	8,630,214.79	939.2	1,337,391.20	1,156,191.92	97.1	2,627,483.66	1,402,557.40	66.4	3,681,111.72	1,998,563.84	24.9	Oklahoma	
Oregon	61,350,150.80	21,960,732.04	1188.6	14,254,516.22	4,159,477.13	303.9	15,315,924.81	4,483,937.88	304.5	3,978,264.56	1,458,553.84	83.3	Oregon	
Pennsylvania	3,988,616.09	1,950,829.06	86.7	1,244,797.29	439,650.00	29.3	778,986.12	205,665.00	13.7	957,110.35	237,810.00	15.8	Pennsylvania	
Rhode Island	15,020,639.90	6,766,322.93	1481.9	1,981,400.03	780,665.87	86.5	6,163,985.71	2,575,506.55	209.2	2,612,449.54	1,081,188.21	21.4	Rhode Island	
South Carolina	17,468,373.19	8,003,826.97	2181.2	1,655,186.12	1,229,074.24	284.2	3,273,534.01	1,717,631.55	576.1	650,258.75	326,014.91	130.6	South Carolina	
South Dakota	21,624,631.57	10,276,584.02	780.0	2,562,769.67	1,039,742.90	78.8	8,316,311.81	3,468,402.56	235.6	111,887.73	35,000.00	0.3	South Dakota	
Tennessee	69,181,673.48	27,440,254.72	4920.2	8,371,957.91	3,821,492.95	531.8	15,931,788.95	6,831,159.89	527.5	1,694,198.26	813,238.16	68.7	Tennessee	
Texas	8,253,178.03	5,098,440.68	546.4	901,199.30	668,639.27	82.5	2,198,402.66	1,672,856.84	161.8	1,824,314.46	83,112.69	11.6	Texas	
Utah	4,242,042.64	2,017,699.51	134.5	754,434.55	315,571.50	17.2	5,501,129.85	564,170.08	25.9	1,156,363.51	422,028.44	26.9	Utah	
Vermont	21,990,249.44	10,385,728.11	1005.5	4,566,420.13	1,987,927.11	149.7	4,846,810.87	1,996,553.04	113.1	1,071,311.26	353,530.29	21.9	Vermont	
Virginia	17,076,511.63	7,782,909.46	688.6	1,105,394.34	463,642.49	42.5	3,690,460.86	2,750,600.00	68.2	273,990.33	50,501.38	10.2	Virginia	
Washington	9,473,716.44	4,141,022.65	392.9	951,130.88	432,685.36	26.5	7,132,862.00	2,934,191.85	238.2	109,976.65	54,988.32	5.7	Washington	
West Virginia	24,865,508.19	10,392,705.73	1592.1	2,768,425.95	1,324,002.42	124.1	6,402,373.50	2,283,441.30	301.6	3,068,668.14	1,379,265.50	86.9	West Virginia	
Wisconsin	10,928,302.56	6,040,887.05	1133.5	1,722,022.81	1,038,390.00	182.4	1,625,170.51	1,025,677.53	104.2	285,445.77	222,920.28	34.5	Wisconsin	
Wyoming				343,664.15	97,440.00	6.5	1,786,323.69	562,362.64	29.7				Wyoming	
HAWAII													HAWAII	
TOTALS	955,632,834.36	426,178,703.58	52,526.6	168,653,102.83	75,541,240.87	7,701.8	318,806,492.63	134,453,457.35	12,543.3	64,117,127.86	25,443,883.76	2,531.7	TOTALS	

* Includes projects reported completed (and vouchers not yet paid) totaling: Estimated cost \$ 71,866,434.08 Federal aid \$ 30,999,665.91 Miles 7,721.6

* Includes projects reported completed (final vouchers not yet paid) totaling: Estimated cost \$71,866,434.08 Federal aid \$30,999,665.91 Miles 2,721.6

PUBLIC ROADS

A JOURNAL OF HIGHWAY RESEARCH



UNITED STATES DEPARTMENT OF AGRICULTURE
BUREAU OF PUBLIC ROADS



VOL. 8, NO. 4



JUNE, 1927



TRAFFIC RECORDING STATION—OHIO TRANSPORTATION SURVEY

PUBLIC ROADS

A JOURNAL OF HIGHWAY RESEARCH

U. S. DEPARTMENT OF AGRICULTURE

BUREAU OF PUBLIC ROADS

CERTIFICATE: By direction of the Secretary of Agriculture, the matter contained herein is published as administrative information and is required for the proper transaction of the public business

VOL. 8, NO. 4

JUNE, 1927

R. E. ROYALL, Editor

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DIGEST OF REPORT OF OHIO HIGHWAY TRANSPORTATION SURVEY

Reported by J. GORDON McKAY, Chief of the Division of Highway Economics, Bureau of Public Roads

THE Report of a Survey of Transportation on the State Highway System of Ohio has recently come from the press.¹ This report contains the results of highway-traffic studies of the State, county, and township road systems of Ohio conducted during 1925 under a cooperative agreement between the Bureau of Public Roads, United States Department of Agriculture, and the Ohio State Department of Highways and Public Works.

The investigation was undertaken in order to obtain essential facts concerning traffic on Ohio highways as a basis for planning the development of the Ohio State highway system to serve present and future traffic.

The conclusions are based upon the present density, type, loading, and distribution of traffic units and a traffic classification of State highways, upon present population and population trends, upon predicted future traffic, and upon an economic and physical analysis of other factors affecting the planning of a program of highway improvement.

The first section of the full report contains a summary of principal conclusions, the second contains the detailed data, and the third section the proposed plan of State-highway improvement indorsed by the Ohio State Department of Highways and Public Works and the United States Bureau of Public Roads.

The highway-traffic studies upon which the report is based were conducted under the joint supervision of Thomas H. MacDonald, Chief of the Bureau of Public Roads, and George F. Schlesinger, director, and L. A. Boulay, former director of the Ohio State Department of Highways and Public Works. J. Gordon McKay, Chief of the Division of Highway Economics, Bureau of Public Roads, directed the work of the survey and preparation of the report, assisted by O. M. Elvehjem, E. T. Stein, L. E. Peabody, and B. P. Root, all of the Division of Highway Economics, and Harry J. Kirk, State highway engineer, and Harry E. Neal, traffic engineer of Ohio.

Of the approximately 11,000 miles of State highways on January 1, 1926, there were 5,194 miles that were improved with brick, concrete, asphalt, and bituminous-macadam surfaces, of which more than half had surfaces less than 18 feet in width, and approximately 400 miles were old and worn-out pavement under county maintenance. Surface-treated macadam surfaces, of which there were 1,307 miles in 1925, will require re-treatment or reconstruction when the traffic materially increases. The remainder comprised 3,000 miles of gravel, slag, and stone and 1,282 miles of unimproved highways.

The principal problems now confronting the State are the reconstruction of the old, worn-out surfaces, expensive to maintain, the widening of narrow pavements, the improvement of unimproved sections of the State system, the elimination of the most dangerous railroad grade crossings, and the distribution of present highway revenues satisfactorily to complete the State system of roads.

Anticipating the need for a definite program of reconstruction and development, and realizing the necessity of having as a basis for a sound plan of highway improvement accurate data with respect to the traffic on the various sections of the State highway system, the Director of Highways and Public Works entered into an agreement with the United States Bureau of Public Roads to conduct a cooperative survey of transportation on the highways of the State.



AT THE CITY LIMITS OF ZANESVILLE ON THE NATIONAL ROAD IN 1913

CONCLUSIONS FROM SURVEY SUMMARIZED

The results of the survey show that during the next five years the State should reconstruct 1,220 miles of the State system, widen 1,594 miles, and build 1,707 miles, the latter comprising 1,007 miles of construction superior to gravel and 700 miles of traffic-bound improvements. The cost is estimated at \$100,000,000.

On the basis of the traffic observed during the survey, it is estimated that the State highway system, which comprises 13 per cent of the total rural mileage, provided highway service for a traffic of 2,160,435,000 vehicle-miles, equal to 57.7 per cent of the total motor-vehicle traffic on the rural highways of the State in 1925; that the county highways, which include 27.1 per cent of the rural mileage, provided service for 1,108,870,000 vehicle-miles, 29.6 per cent of the total traffic; and that the township highways, which constitute 59.9 per cent of rural mileage, provided service for 477,055,000 vehicle-miles, or only 12.7 per cent of total rural traffic. The daily traffic on the State system averages over nine times that upon the county and township roads. These facts show the necessity of allotting a sufficient portion of total highway revenues to complete the improvement of the State system of highways.

It is clearly shown by the survey that the principal routes of the State system, comprising what are known as the Federal-aid and main market systems, are with a few exceptions the most important traffic routes of the State. The improvement of these routes, however, has not up to this time been entirely consistent with their traffic importance. There are sections of gravel and

¹ Copies of the full report may be obtained upon request from the Bureau of Public Roads.

old, worn-out surfaces, narrow 9 to 16 foot pavements, as well as unimproved sections on these heavily traveled routes.

The largest volume of motor-vehicle traffic is found in the areas adjacent to large centers of urban population and on the main, through-traffic routes.

Of the 11,000 miles of the State highway system, 1.2 per cent carried 2,500 or more motor vehicles per day, 7.8 per cent carried 1,500 or more, 29.4 per cent carried 600 or more, and 70.6 per cent carried less than 600 vehicles per day in 1925.



PAN AMERICAN HIGHWAY ENGINEERS AND BUSES IN WHICH THEY INSPECTED OHIO HIGHWAYS IN 1924

The northeastern part of the State is the most important traffic area and is also the region of densest population, motor-vehicle registration, and industrial development. The southwestern area is very close to the northeastern section in traffic importance. The northwestern and southeastern parts of the State are of least present and expected future traffic importance.

Motor-truck traffic is an important part of total traffic on the principal routes. On the State system in 1925, 2.7 per cent of the mileage carried 200 or more trucks per day, 5.7 per cent carried 150 or more, 13.1 per cent 100 or more, and 27.5 per cent carried 60 or more trucks per day. Based on the traffic forecast, it is expected that by 1935 over 1,300 miles of State highways will carry 200 or more trucks per day, while a comparatively small mileage is expected to carry from 500 to 1,000 trucks a day.

Four-fifths of the trucks operating on the State system are small units, $2\frac{1}{2}$ tons capacity or less. The comparatively small number of large-capacity trucks and heavy loads observed may perhaps be attributed in a measure to the gross-load limitation of 20,000 pounds fixed by law in the State.

The average daily density of traffic in 1925 was 538 vehicles on the State system, 132 on county highways, and only 26 on township roads, each mile of road of the State system providing traffic service more than equal to that of 4 miles of county and 20 miles of town roads.

The Federal-aid system, slightly more than half the State system, carried 70.6 per cent of the daily traffic of the State system; the main market roads, approximately one-third of the State-system mileage and included for the most part in the Federal-aid system, carried over half the traffic, and the principal routes of the system, 8.8 per cent of the mileage, carried over one-fourth of the traffic.

Foreign traffic comprises but a small part of the total traffic except on the principal through-traffic routes. More than half of passenger-car traffic consists of cars used for business purposes.

The survey clearly shows that the traffic using the State system is predominantly city passenger-car and motor-truck traffic, farm-owned passenger cars and motor trucks making up only 12.4 per cent and 15.5 per cent, respectively, of the total passenger-car and motor-truck traffic. The improvement and maintenance of State highways is, therefore, primarily the result of the demand for highway service by city motor-vehicle owners.

The volume of traffic in a given area is principally produced by the population residing within a radius of 30 miles, since less than 30 per cent of the truck traffic and less than 40 per cent of the passenger-car traffic travels more than 30 miles.

FUTURE TRAFFIC AND HIGHWAY REQUIREMENTS ESTIMATED

The distribution of population is an important factor in planning highway improvements. Of the entire area of the State 80 per cent has a population of less than 80 persons per square mile and is the home of only 22.7 per cent of the total State population, whereas the 9.4 per cent of the area that has a population of 160 or more persons per square mile includes 69.3 per cent of the population of the State.

These variations indicate marked differences in the necessity for highway service in the several parts of the State. In the densely populated areas the highway system should be planned to serve large volumes of traffic between the principal centers of population with tributary feeder routes connecting minor population centers with the primary traffic routes. These routes should be of sufficient width and improved with surfaces adequate to carry the large daily volume of traffic as directly as possible; obstructions to the free movement of traffic, such as railway crossings at grade, sharp curves, heavy grades, and congested traffic sections should be eliminated, and by-pass routes should be constructed to avoid the traffic congestion which occurs when a main traffic route passes through the business center of small villages and cities. In the sparsely populated areas the volume of traffic is smaller and its sources more scattered. A connected system of main routes comprising a smaller mileage, improved with gravel or the lower types of paved surfaces where traffic warrants such improvements, should satisfactorily meet traffic requirements in these areas, except on the main through routes traversing them. The removal of obstacles to the easy movement of traffic is not an important problem in areas of low population and traffic, particularly when the expectancy of future traffic increase is small.

The density of traffic on the various roads of the State system has been used as the basis for an estimate of traffic on the same roads in 1930 and 1935, applying for this purpose the relation between the increase in traffic on the highways and the ratio of population to motor-vehicle registration observed in other States. In 1925 there was one motor vehicle for each 4.7 persons in Ohio. Extending the past trend of this ratio to 1935 it is estimated that there will then be one vehicle for each 2.82 persons. On this basis the registration of 1935 is estimated at 2,607,000 motor vehicles, a registration approximately twice as great as that of 1925. As the yearly increase of motor-vehicle traffic on the highways has been found to be practically in direct proportion to the growth of motor-vehicle registration, it may be expected that traffic on the State highways will increase 51 per cent between 1925 and 1930, and 28 per cent between 1930 and 1935.



TYPES OF ROAD CONSTRUCTED ON THE OHIO STATE SYSTEM IN RECENT YEARS

As a basis for the plan of highway improvement, the State highways are classified in three groups designated as major, medium, and minor traffic highways, according to their average daily traffic. Routes or sections of routes carrying 1,500 or more motor vehicles per day are classed as major routes, those carrying 600 to 1,500 vehicles per day as medium routes, and those carrying less than 600 vehicles daily are classed as minor routes. The routes or sections of routes are classed in this way on the basis of the observed 1925 traffic, and the estimated traffic for 1930 and 1935 is employed in a similar manner to indicate the probable classification in those years.

Experience in many States indicates that ordinary untreated gravel and similar surfaces can not be economically maintained when the traffic exceeds 500 to 600 vehicles per day, and similar experience in Ohio points to approximately 600 vehicles per day as the limit. Above that traffic density the type and design of surface required are largely functions of the frequency of heavy loads, the choice of types including bituminous macadam for the lower densities and the several rigid types for roads of greater density.

If, on the basis of this experience, those sections of the Ohio State system which carry a traffic of 600 or more vehicles per day be considered as requiring a type of surface superior to untreated gravel, it is found that in 1925 over one-third of the 11,000 miles of the State system, or 3,852 miles, required such surfaces, and 10 years later, in 1935, based on the estimated traffic, approximately half the system, or 5,221 miles, should be so improved.

THE METHODS OF THE SURVEY

The traffic survey was begun in December, 1924, and continued for a period of one year. During the survey traffic data were recorded at 1,158 points on Ohio highways. At 358 of these points complete data were recorded one day each month during the year period. At the remaining 800 points counts of passenger cars and motor trucks were obtained on three days during the summer months. Data obtained included counts of passenger cars, motor trucks, motor busses, horse-drawn vehicles, foreign vehicles, and detailed truck and passenger-car data. Motor-truck data included the capacity of the truck, State of registration, place

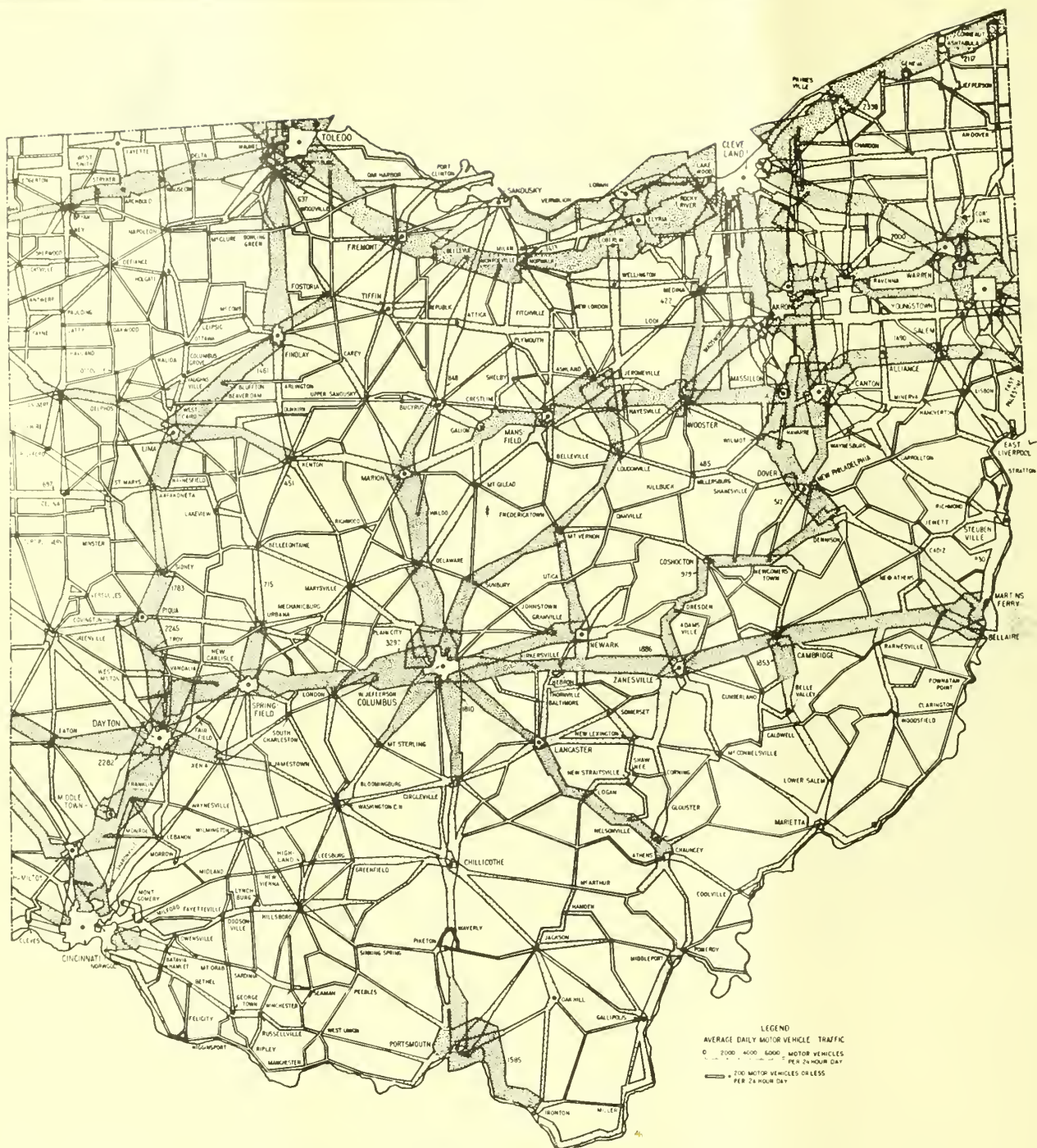


FIG. 1.—AVERAGE DAILY DENSITY OF MOTOR VEHICLE TRAFFIC ON THE STATE HIGHWAY SYSTEM

of ownership, origin, destination, type of origin and destination, commodity carried, and tire equipment. At alternate operations at 156 stations total gross and rear-axle weights were measured by means of portable scales. Passenger-car data included State of registration, situs of ownership, purpose of trip, origin, destination, and number of passengers.

Each operation consisted of a 10-hour observation period, alternating between 6 a. m. to 4 p. m. and 10 a. m. to 8 p. m. Special observers tabulated traffic between 8 p. m. and 6 a. m. at selected stations. Complete 24-hour observations were therefore available at these stations, which were made the basis of computation of hourly variations in traffic and of average daily

traffic at all stations. Traffic observations for week periods were also made at selected stations to determine variations in traffic by days of the week. Seasonal variations were computed from the monthly operations at all stations. Stations were operated on a carefully planned schedule which permitted operations on the various days of the week and prevented duplicate recording of traffic.

Traffic was observed on practically all sections of the State highway system, and stations were so located as to enable close observation of the variations in traffic on various routes and sections of routes. Stations were also located on representative sections of the county and township highway systems in all sections of the State.

STATE SYSTEM CARRIES LARGE PERCENTAGE OF TOTAL TRAFFIC

There are in Ohio approximately 84,884 miles of rural highway, of which, on January 1, 1926, 11,000 miles were intercounty highways, constituting the State highway system. Of the remaining 73,884 miles, 22,991 were included in the county system, and 50,893 were township roads. Federal-aid highways included in the State highway system aggregated 5,899 miles and the main market road system 3,486 miles.

Upon the 84,884 miles it is estimated that in 1925 there was a motor-vehicle movement of approximately 3,746,360,000 vehicle-miles. The relative importance of the State highway system is indicated by the fact that although it includes but 13 per cent of the entire rural road mileage, it carried 2,160,435,000 vehicle-miles, or 57.7 per cent, of the total motor-vehicle traffic during 1925. The average daily motor-vehicle traffic per mile upon the State highway system is over nine times that upon the county and township roads.

The daily volume of traffic on different parts of the State highway system varies widely. The number of motor vehicles per average 24 hour day varied from 5,583 on route U. S. 30, between Canton and Massillon, to a minimum of less than 20 vehicles on several unimproved sections. The State highway system includes 4,180 miles, on which the average daily motor-vehicle traffic per mile was less than 200 vehicles. County and town roads include a considerable mileage, on which the average daily traffic was less than 5 vehicles per day.



NORTH HILL VIADUCT AT AKRON, OHIO. LENGTH 2,800 FEET; ROADWAY, 52 FEET; BUILT 1921

important traffic routes of the State are apparent. Figure 2 shows the mileage of State highways by traffic density classes.

PRINCIPAL TRAFFIC ROUTES DISCUSSED

The largest volume of traffic of both passenger cars and trucks is found in the areas adjacent to large centers of urban population and on the main through routes. The concentration of traffic in the areas immediately adjacent to the larger cities of the State is apparent. The principal through routes are also clearly evident as broad bands serving wide areas in the State and connecting the important cities of Ohio and near-by States. The traffic importance of these principal routes is the result of local traffic of the area augmented by the traffic moving between larger centers of population. Among the more important of the through routes are the following: The Buffalo-Chicago Highway, crossing Ohio near its northern border, connecting the Buffalo and Erie territory with Cleveland and surrounding cities, Toledo and other points in Ohio, and cities in Indiana, Michigan, and Illinois; the National Pike, from Bridgeport through Zanesville, Columbus, Springfield, and Dayton,² and connecting Wheeling and eastern points with central Ohio cities and with cities in Indiana, Illinois, and points west; the Lincoln-Harding Highway (route U. S. 30), through Canton, Mansfield, Marion, and Lima, connecting Pittsburgh and Pennsylvania cities with the above cities in Ohio and with Indiana and Illinois cities to the west. Crossing the State in a north and south direction are the Dixie Highway, from Toledo through Dayton to Cincinnati, connecting Detroit and other Michigan cities with areas south of Ohio; the Scioto Trail, from Sandusky, via Marion and Columbus, to Portsmouth; the Cincinnati-Columbus-Cleveland (the "C. C. C.") Highway; and the Cleveland-Marietta Highway, via Akron, Canton, New Philadelphia, and Cambridge.

² The National Pike is routed north of Dayton, but due to the condition of sections of this route west of Brandt traffic on the route passes through Dayton.

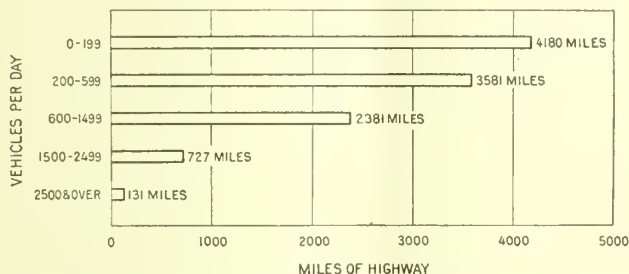


FIG. 2.—MILEAGE OF STATE HIGHWAYS BY TRAFFIC DENSITY CLASSES

During 1925 the density of horse-drawn vehicles was recorded at all survey stations, but it was early apparent that their numbers were so few as to warrant no consideration in highway planning. The average traffic of horse-drawn vehicles on State highways is less than 7 per day.

Motor-bus traffic is important on several of the State routes. It is, however, a specialized movement, and its volume on any highway is the product of several factors which have little effect upon other motor-vehicle traffic. Motor-bus traffic is, therefore, discussed separately in the report, and the term "motor-vehicle traffic" refers only to passenger cars and motor trucks.

The complete report shows the traffic density at each of the 1,158 points where traffic was observed during the survey and shows also, by counties, the routes upon which traffic was observed, the average density of motor-vehicle traffic for a 24-hour day, the average daily density of motor-truck traffic, the normal maximum traffic, and the estimated traffic for 1930.

The average daily distribution of total traffic on the State highway system is shown in Figure 1. The more

The principal through routes coincide in general with the routes adopted for uniform marking by the American Association of State Highway Officials in November, 1926, referred to as United States numbered routes, of which the most important east and west routes are U. S. 20, 30, 40, and 50, and the most important north and south, U. S. 21, 23, and 25.

Route U. S. 20 from the Pennsylvania line follows the Buffalo-Chicago Highway west through Ashtabula, Cleveland, Elyria, Norwalk, and Fremont. A few miles west of Fremont it diverges from the present heavily traveled route and continues through Perrysburg and Maumee rather than Toledo, thence north, crossing State route 2 (the present principal traffic route) and west to the Indiana line over a route at present carrying very light traffic. From Conneaut to Fremont traffic on this route averages 2,447 vehicles per day; from Maumee to the Indiana line but 297. When proposed improvements on this portion of route U. S. 20 are completed, through traffic will to a larger extent use this route in preference to route 2.



A TRUCK AND TRAILER COMBINATION

Route U. S. 30 follows the Lincoln and Harding Highways across the State, from East Liverpool through Lisbon, Canton, Mansfield, Marion, Lima, and Van Wert. This route is already an important cross-State route, although east of Canton through traffic has followed the route via Salem and East Palestine rather than the route via Lisbon and East Liverpool, as the former provided a more completely improved route to Pittsburgh. Average traffic for the entire length of route U. S. 30 is 1,071 vehicles per day.

Route U. S. 40, the National Pike, from Wheeling, W. Va., passes almost due west through Zanesville, Columbus, and Springfield to the Indiana line. The western end of the route is unimproved at present, with the result that through traffic detours via Dayton and Eaton. Improvement of less than 20 miles in Preble County will open route U. S. 40 as a direct bypass for through traffic north of Dayton. Traffic in 1925 averaged 1,749 vehicles per day on the 159 miles from Brandt east to the Pennsylvania line.

Route U. S. 50 crosses the State from Belpre via Athens, McArthur, Chillicothe, Hillsboro, and Cincinnati to the Indiana line and is one of the less important United States routes, averaging but 452 vehicles per day. It passes through few large centers of population and for much of its length is surfaced with gravel.

Route U. S. 25, the Dixie Highway from the Michigan line north of Toledo to Cincinnati, is the most important north and south through route and is paved throughout practically its whole length. South of Franklin, in Warren and Butler Counties, the new route follows the most direct course between Dayton and Cincinnati. For 136 miles north of Franklin the average traffic is 1,743 vehicles per day.

Route U. S. 23 crosses the State from the Michigan line, north of Toledo, to Portsmouth via Fostoria, Marion, Columbus, and Circleville. South of Marion it follows the Scioto Trail. On account of the relative lack of improvement in its northern portion, as compared with the Dixie Highway to Findlay, the traffic north of Carey is very light. Between Marion and Columbus traffic averaged 2,160, and between Columbus and Portsmouth it averaged 978 vehicles per day.

Route U. S. 21, from Cleveland to Marietta, coincides with the old Cleveland-Marietta highway south of Newcomerstown and is located west of Canton and Akron to avoid the congested urban traffic of this area.

The system of numbered United States highways in Ohio when improved will form a well-balanced network of the more important through-traffic highways of the State. Many of these routes will serve a large volume of local traffic. The total traffic on each route will depend very largely upon the population and development of the immediate areas which it traverses, and traffic upon those routes which pass through the sparsely populated sections of the State will continue to be small as compared with those routes which connect the important sources of local traffic.

Of the 11,000 miles of the State highway system, 131 miles, or 1.2 per cent of the total mileage, carried 2,500 or more motor vehicles per day in 1925; 858 miles, 7.8 per cent of the system, carried 1,500 or more; 3,239 miles, approximately 30 per cent of the total, carried 600 or more; and 7,761 miles, 70.6 per cent, carried less than 600 vehicles per day, of which 4,180 miles carried less than 200, as shown in Figure 2.

The routes carrying the largest daily volume of traffic are with few exceptions in the northeastern, northern, and southwestern parts of the State, and the routes of least traffic importance are in the southeastern and northwestern sections.

On the basis of traffic the State is divided into five separate traffic sections, each of these sections being subdivided in the order of their traffic importance into two or more divisions, somewhat comparable with the distribution of population and industry. Table 1 shows the mileage of State highways, by traffic classes, in the five traffic sections.

TABLE 1.—Mileage of State highways, by traffic classes, in the five traffic sections

Section	All State highways		Daily traffic, over 1,500 vehicles		Daily traffic, 600 to 1,500 vehicles		Daily traffic, less than 600 vehicles	
	Miles	Per cent of total miles	Miles	Per cent of total miles	Miles	Per cent of total miles	Miles	Per cent of total miles
Northeastern.....	2,821	25.6	454	52.9	836	35.1	1,536	19.8
Southwestern.....	736	6.7	133	15.5	202	8.5	401	5.2
East-central.....	1,285	11.7	86	10.0	265	11.1	934	12.0
Northwestern.....	3,402	30.9	162	18.9	679	28.5	2,556	32.9
Southern.....	2,756	25.1	23	2.7	399	16.8	2,334	30.1
State total....	11,000	100.0	858	100.0	2,381	100.0	7,761	100.0

TRUCK TRAFFIC AN IMPORTANT FACTOR

Although motor-truck traffic on the State highway system is only 9.5 per cent of total motor-vehicle traffic measured in vehicle-miles, motor-truck traffic is an important factor in highway-traffic planning.

The average gross weight of motor trucks using the State system is over twice that of passenger cars, while maximum motor-truck weights are four times the maximum weights of passenger cars. The importance of the motor truck is further emphasized by the fact that many trucks are equipped with cushion and solid tires, which are much less effective in cushioning the impact of the wheels than the pneumatic tires with which the passenger cars are equipped. A study of the rear-wheel tire equipment of motor trucks using the State highway system shows that 15 per cent of all the trucks are equipped with solid tires and a like percentage with cushion tires, and that 95 per cent of the 3 to 7½ ton trucks are equipped with cushion or solid tires on the rear wheels.

Since motor trucks carry heavier gross loads than passenger cars, and the larger capacities are equipped with solid or cushion tires, and because the trucks do not have the refinements in shock-absorbing devices and spring equipment possessed by the passenger cars, the motor truck, where it forms an appreciable part of motor-vehicle traffic, presents a special problem for the highway builder.

The average daily density of motor-truck traffic varies greatly on different routes and in various parts of the State. Of the 11,000 miles of State highways, 299 miles, 2.7 per cent of the total, carried in 1925, 200 or more trucks per day; 629 miles, 5.7 per cent, carried 150 or more; 1,442 miles, 13.1 per cent, carried 100 or more; 3,019 miles, 27.5 per cent, carried 60 or more; and 7,981 miles, 72.5 per cent, carried less than 60 trucks, of which 5,305 miles, 48.2 per cent, carried less than 30, as shown in Figure 3. On routes carrying a small average daily truck traffic, especially those on which there are less than 30 trucks, and many sections on which the daily density ranges from 30 to 59 class the number of trucks is practically negligible in planning highway improvements. An improvement sufficient for passenger-car traffic will, with but few exceptions, prove satisfactory

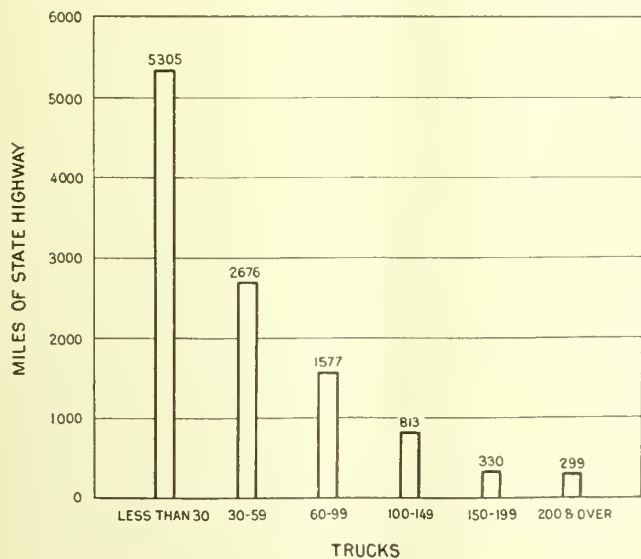


FIG. 3.—MILEAGE OF STATE HIGHWAYS CARRYING VARIOUS NUMBERS OF MOTOR TRUCKS PER DAY

for the small-capacity trucks using these routes. On those routes carrying 60 or more, and particularly the routes carrying 100 or more, the motor truck becomes an important factor in planning the improvement of the highways.

Sections of highway which have a traffic of more than 200 trucks per day include 299 miles, or 2.7 per cent, of the total State highway mileage. On the basis of the traffic forecast there will be 841 miles of State road in 1930 and 1,351 miles in 1935, on which the motor-truck traffic density will exceed 200 trucks per day.

U. S. route 30, Massillon to Canton, carries an average of 485 trucks per day, which is the highest truck density on any section in the State. In the forecast for 1935, truck traffic on this route is estimated at 1,000 per day. Highway sections carrying from 150 to 200 trucks per day comprise 330 miles, or 3 per cent, of the total mileage of State highways.



MILK AND OTHER DAIRY PRODUCTS ARE AMONG THE PRINCIPAL COMMODITIES HAULED BY TRUCKS

CITIES INFLUENCE TRUCK TRAFFIC

The more important of the five traffic sections shown in Table 1, from the standpoint of motor-truck density, are the northeastern section, where the average daily density of motor trucks is 77 per mile of State highway, and the southwestern section with a density of 75 trucks. The corresponding density is 53 in the east-central section, and 36 in the northwestern and southern sections.

The comparative importance of the northeastern section is further emphasized by the fact that this section includes 25 per cent of the total mileage of State highways. The southwestern section, although of almost equal truck traffic density, includes only 7 per cent of the mileage of State highways. The northwestern and southern sections, with a density of only 36 trucks per day, include 56 per cent of the State-highway mileage.

The size, location, and industrial development of cities and towns determines very largely the volume of motor trucking on routes in the several sections of the State. Centers of population and industry are the main source and destination of goods transported by motor truck, and the principal trucking routes are those serving the territory adjacent to the larger cities and those routes connecting centers of population.

The influence of large cities on motor-truck traffic is clearly shown by the data collected. The nine largest cities in the State, in order of population, are Cleveland, Cincinnati, Toledo, Columbus, Akron, Dayton, Youngstown, Canton, and Springfield. Around these cities

with one or two exceptions, is found the greatest volume of motor-truck traffic. The influence of large cities upon truck traffic in the five sections of the State is shown in Table 2.

It is apparent that the high density of truck traffic per mile on the State roads of the northeastern and southwestern sections is caused by the large cities in these areas. The two sections, which together include 30 per cent of the total area of the State, include 58 per cent of the cities of over 10,000 population. The northwestern and southern sections, the least important regions of truck traffic, contain 58 per cent of the area and include only 24 per cent of the cities of over 10,000 population.



A TYPICAL ROADSIDE MARKET

The more important trucking areas, the northeastern and southwestern sections, have five of the seven cities between 30,000 and 100,000 population, and six of the seven cities of over 100,000 population. Cleveland in the northeast and Cincinnati in the southwest are the predominating traffic influences in these two regions.

The motor-truck registration per square mile, shown in Table 2, indicates the concentration of trucks in the southwestern and northeastern sections. The comparatively low registration per mile in the three remaining sections, especially in the southern section, again reflects the relatively small number of large centers of population and industry and accounts for the lower truck-traffic density in these areas.

Figure 4 shows the distribution of motor trucks by capacity groups, Figure 5 the distribution of loaded motor trucks by gross-weight groups, Figure 6 the mileage of State highways carrying various numbers of 3 to 7½ ton trucks, and Figure 7 the distribution of motor trucks by capacity classes in the five traffic sections of the State.

TABLE 2.—Average truck-traffic density on State roads in the five traffic sections of the State compared with the relative area of the sections, their truck registration per square mile, and the number of cities over 10,000 population in each

Section	Average truck-traffic density	Percentage of total area in the State	Truck registration per square mile (1924)	Cities over 10,000 population, by population classes ¹				
				Total		10,000 to 30,000	30,000 to 100,000	Over 100,000
				Number	Per cent			
Northeastern.....	77	24	7.9	22	44	15	3	4
Southwestern.....	75	6	10.9	7	14	3	2	2
East-central.....	53	12	2.5	9	18	9	—	—
Northwestern.....	36	30	2.3	7	14	5	1	1
Southern.....	36	28	1.3	5	10	4	1	—
Total.....	51	100	3.9	50	100	36	7	7

¹ United States census of 1920.

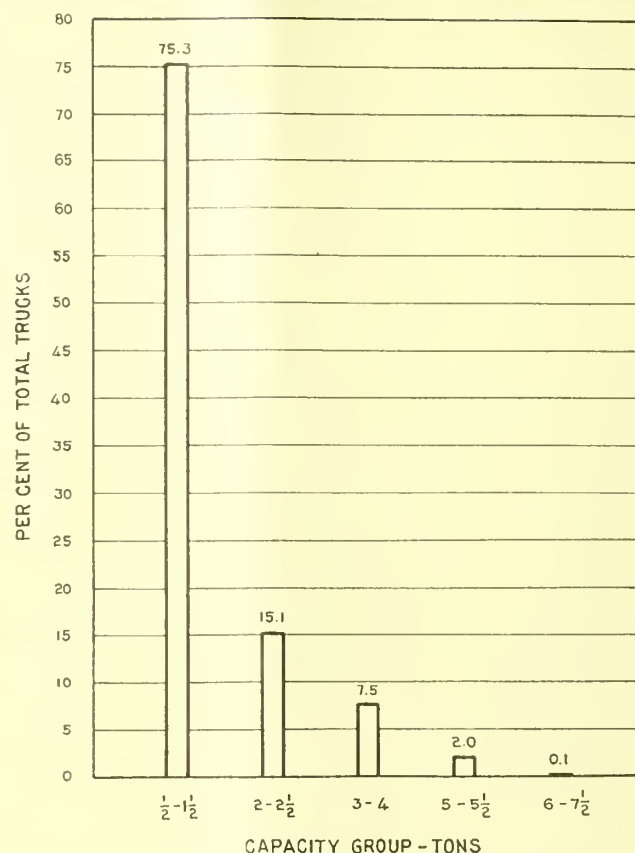


FIG. 4.—DISTRIBUTION OF MOTOR TRUCKS BY CAPACITY GROUPS

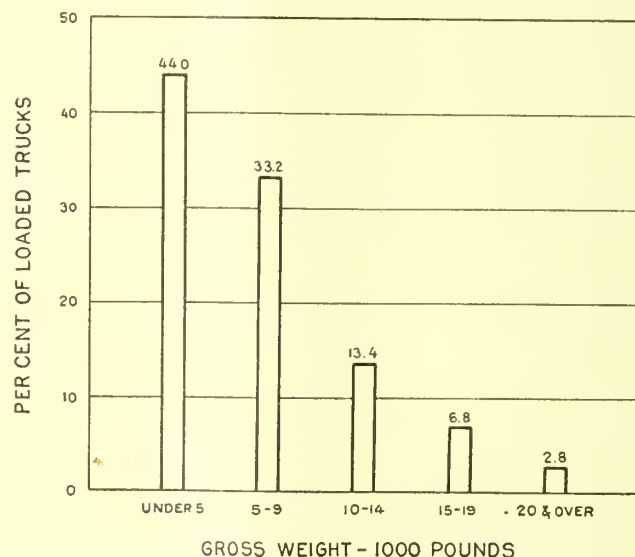


FIG. 5.—DISTRIBUTION OF LOADED MOTOR TRUCKS BY GROSS-WEIGHT GROUPS

It is possible that the high density of motor-truck traffic in the important traffic areas of the State will eventually present problems difficult of solution. Routes having a large number of motor trucks also carry a large number of passenger cars. On such routes, unless supplementary highway facilities are provided, serious congestion problems will develop. The mileage subject to this danger is not large. In 1925 there were only 15 miles of State highway on

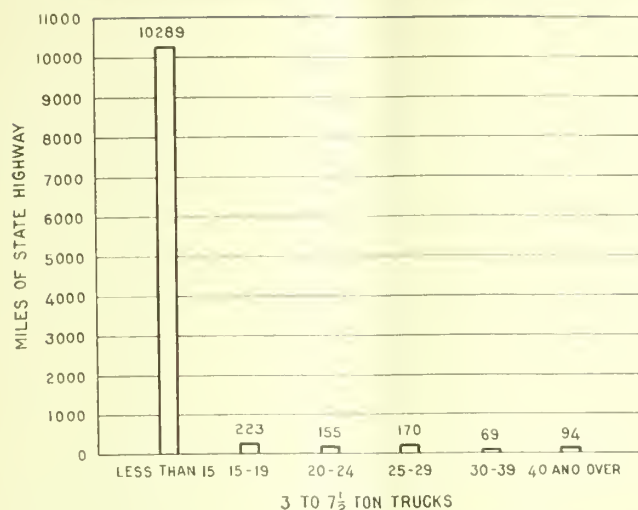


FIG. 6.—MILEAGE OF STATE HIGHWAYS CARRYING VARIOUS NUMBERS OF 3 TO 7½ TON TRUCKS

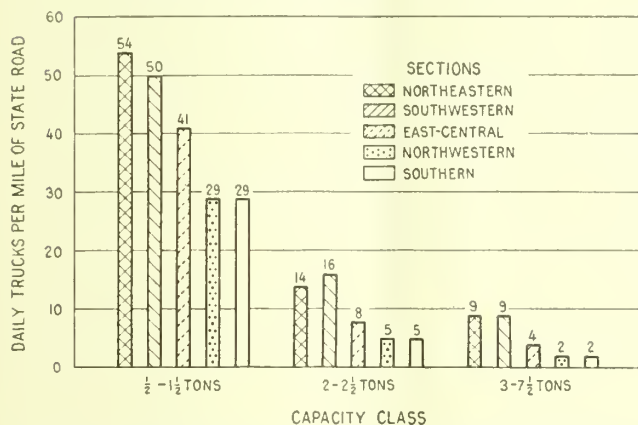


FIG. 7.—DISTRIBUTION OF MOTOR TRUCKS BY CAPACITY CLASSES IN THE FIVE TRAFFIC SECTIONS OF THE STATE

which the density of motor-truck traffic exceeded 400 trucks per day and only 87 miles where the daily density was greater than 300 trucks.

The relief of congestion which will probably result on a few sections of the State system undoubtedly lies in increasing the width of road and creating separate lanes for truck traffic or in the construction of parallel routes. In either case the segregation of passenger-car and motor-truck traffic would speed up passenger-car traffic and eliminate congestion caused by heavy, slow-moving vehicles.

HIGHWAY UTILIZATION

During 1925 traffic of passenger cars and trucks on the 84,884 miles of rural highways in Ohio was approximately 3,746,360,000 vehicle-miles. The distribution of traffic varies greatly on the several highway systems, on sections of each system, and in the several traffic areas of the State. The three classes into which the rural highways of the State are divided are State highways, of which there are 11,000 miles, county highways, which total 22,991 miles, and township highways, of which there are 50,893 miles. The distribution of vehicle mileage on each of these systems or their traffic use is shown in Table 3 and Figure 8.

The State highway system, 13 per cent of the rural highway mileage, carries 57.7 per cent of the traffic

TABLE 3.—Motor-vehicle utilization and mileage of Ohio rural highways, by systems¹

Highway system	Mileage	Per cent of total mileage	Daily vehicle-miles	Annual vehicle-miles	Per cent of total vehicle-miles	Average daily traffic density
State highways.....	11,000	13.0	5,919,000	2,160,435,000	57.7	538
County highways.....	22,991	27.1	3,038,000	1,108,870,000	29.6	132
Township highways.....	50,893	59.9	1,307,000	477,055,000	12.7	26
Total.....	84,884	100.0	10,264,000	3,746,360,000	100.0	

¹ Motor vehicles refer to passenger cars and trucks only, excluding motor busses.

measured in vehicle-miles. Contrasted with this system is the township highway system with 59.9 per cent of the highway mileage and only 12.7 per cent of the vehicle mileage.

The average daily density of traffic on the State system is 538 vehicles, on the county system 132, and on township roads only 26.

The distribution of highway mileage and vehicle mileage by highway systems in each of the five sections of the State is shown in Table 4.

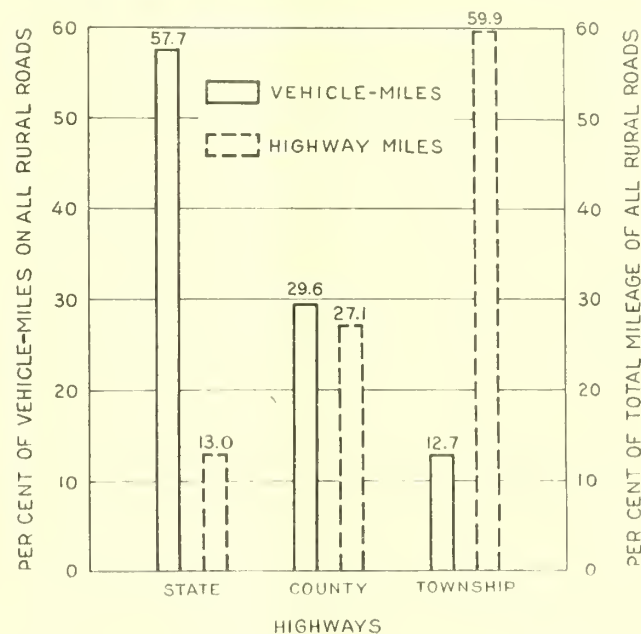


FIG. 8.—COMPARISON OF TRAFFIC ON THE STATE, COUNTY, AND TOWNSHIP HIGHWAY SYSTEMS, AND THE PROPORTIONATE MILEAGE OF THE THREE SYSTEMS

The State highways in the northeastern section of the State, comprising 25.6 per cent of the State highway mileage, carry 38.3 per cent of the total traffic on the State highway system. In the southwestern section there is 6.7 per cent of the State highway mileage which carries 9.6 per cent of total traffic on the system. In contrast with these areas is the southern section, with 25.1 per cent of the State highway mileage and only 16.1 per cent of the total traffic on the system. Variations in traffic on the county and township system in the five regions are almost equally pronounced.

Traffic on the State highway system also varies greatly on the different routes, ranging from more than 5,500 vehicles per day on the heaviest route observed to less than 50 vehicles per day. The daily use of selected sections of the State highway system is shown in Table 5 and Figure 9.

TABLE 4.—Motor-vehicle utilization and mileage of Ohio rural highways in the five traffic sections

Highway systems and sections of State	Mileage of highways	Per cent of highway mileage	Daily vehicle-miles	Per cent of daily vehicle-miles	Average daily density of traffic
State highways:					
Northeastern	2,821	25.6	2,268,000	38.3	804
Southwestern	736	6.7	571,000	9.6	776
East-central	1,285	11.7	650,000	11.0	506
Northwestern	3,402	30.9	1,478,000	25.0	434
Southern	2,756	25.1	952,000	16.1	345
Total	11,000	100.0	5,919,000	100.0	538
County highways:					
Northeastern	5,916	25.7	1,128,000	37.1	191
Southwestern	1,788	7.8	353,000	11.6	197
East-central	2,654	11.6	319,000	10.5	120
Northwestern	7,547	32.8	814,000	26.8	108
Southern	5,086	22.1	424,000	14.0	83
Total	22,991	100.0	3,038,000	100.0	132
Township highways:					
Northeastern	10,625	20.9	428,000	32.8	40
Southwestern	2,579	5.1	102,000	7.8	39
East-central	7,372	14.5	184,000	14.1	25
Northwestern	14,889	29.2	326,000	24.9	22
Southern	15,428	30.3	267,000	20.4	17
Total	50,893	100.0	1,307,000	100.0	26
All highways:					
Northeastern	19,362	22.8	3,824,000	37.3	197
Southwestern	5,103	6.0	1,026,000	10.0	201
East-central	11,311	13.3	1,153,000	11.2	102
Northwestern	25,838	30.5	2,618,000	25.5	101
Southern	23,270	27.4	1,643,000	16.0	71
State total	84,884	100.0	10,264,000	100.0	121

TABLE 5.—Average daily traffic on selected sections of the State highway system

Sections of system	Highway miles	Per cent of State highway system	Per cent of total rural highway system	Average daily vehicle-miles	Per cent of daily vehicle-miles on State highway system	Per cent of daily vehicle-miles on all rural highways	Average daily density of traffic
Federal-aid system ¹	5,899	53.6	6.9	4,177,000	70.6	40.7	708
United States highways	1,878	17.1	2.2	1,848,000	31.2	18.0	984
Main market roads	3,486	31.7	4.1	3,158,000	53.4	30.8	906
Selected principal routes	970	8.8	1.1	1,645,000	27.8	16.0	1,696
State highway system	11,000	100.0	13.0	5,919,000	100.0	57.7	538

¹ With few exceptions the Federal-aid system includes all United States highways and main market highways as well as the roads designated as principal routes.

The Federal-aid system, slightly more than half the State system, carries 70.6 per cent of the daily traffic; main market roads, approximately one-third the mileage, carry over half the traffic; and the principal routes, 8.8 per cent of the mileage, carry over one-fourth of the traffic. These divisions of the State highway system indicate clearly the variation in traffic on this system.

The distribution of passenger-car and motor-truck traffic on the different highway systems is shown in Table 6.

TABLE 6.—Motor-truck and passenger-car utilization of Ohio rural highways, by systems

System	Daily truck-miles	Per cent of total truck-miles	Daily passenger-car miles	Per cent of total passenger-car miles	Ratio of truck-miles to total vehicle-miles
State highways	565,000	54.1	5,354,000	58.1	9.5
County highways	332,000	31.8	2,706,000	29.3	10.9
Township highways	147,000	14.1	1,160,000	12.6	11.2
Total	1,044,000	100.0	9,220,000	100.0	10.2

Motor-truck traffic on the township system, and to a lesser degree on the county system, is almost exclusively made up of small-capacity trucks, while on the State system is included a larger proportion of medium and large capacity trucks.

There is also considerable variation in the relative number of passenger cars and trucks on different routes of the State highway system. Motor trucks vary from less than 6 per cent to more than 20 per cent of total number of vehicles. The extremes of the range are found on routes of minor traffic importance; on nearly all the important traffic routes the percentage falls between 7 and 11. Passenger cars are found to be relatively more important on the principal through routes, and the greatest proportion of trucks is found on important routes in the industrial areas which are not a part of these through routes.

FORECAST OF HIGHWAY TRAFFIC

A knowledge of future traffic, in so far as it can be predicted with reasonable accuracy, is essential in the establishment of a sound plan of highway improvement. A forecast based directly upon past traffic trends is not possible, since there is no historical series of highway-traffic records in Ohio. Such records are available in

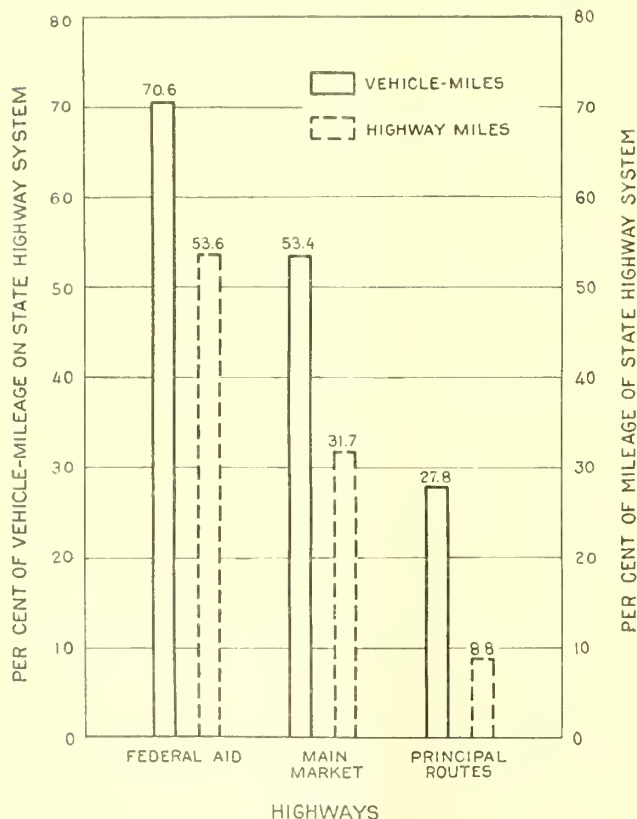


FIG. 9. PERCENTAGES OF THE TOTAL VEHICLE MILEAGE ON THE INTERCOUNTY SYSTEM THAT IS CARRIED BY THE FEDERAL-AID AND MAIN MARKET HIGHWAYS AND BY THE PRINCIPAL ROUTES OF THE STATE, AND THE MILEAGE OF THE SEVERAL CLASSES OF HIGHWAYS

the States of Maine, Maryland, Massachusetts, Michigan, and Wisconsin. Highway traffic and motor-vehicle registration in each of these States have increased at approximately equal rates, as shown in Figure 10. The great variations in industrial and agricultural development, in population, in motor-vehicles registration, and in the period of the series apparently have had no effect upon the relationship between the rates of traffic increase and motor-vehicle registration growth.

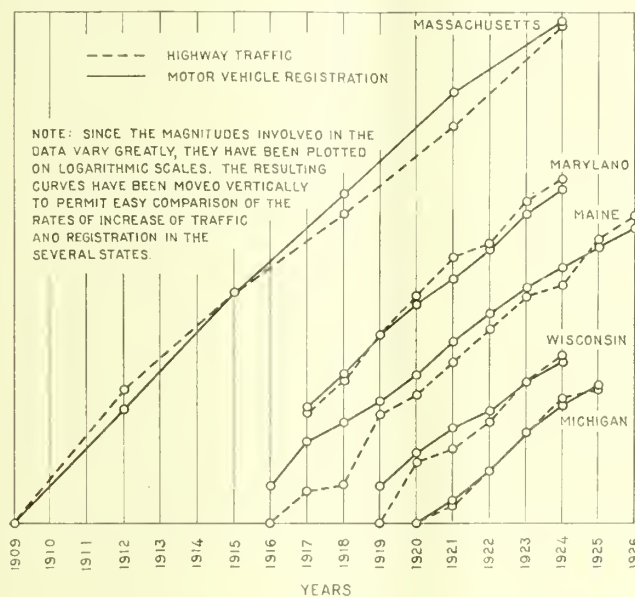


FIG. 10.—TREND OF HIGHWAY TRAFFIC AND MOTOR-VEHICLE REGISTRATION IN MASSACHUSETTS, MARYLAND, MAINE, WISCONSIN, AND MICHIGAN

In the absence of any comprehensive historical series of traffic records in Ohio it has therefore been assumed that highway traffic in Ohio is increasing directly with the increase in motor-vehicle registration.

The increase of motor-vehicle registration is a function of two variables: (1) The increase in population, and (2) the increase in ownership and use of motor vehicles in proportion to population, measured by the number of persons per motor vehicle. The past trend of both of these factors may be determined from available records.

Population changes are measured accurately by the decennial census and intercensal estimates made by the Bureau of the Census. Population, by years, as estimated by the Bureau of the Census, from 1913 to 1923 and estimates calculated by extension of the census method for 1924 to 1930 and for 1935 is shown in Table 7.

The growth of motor-vehicle registration in proportion to population, i. e., the decrease in persons per car, appears to follow the same general characteristics as the growth of population, an early growth slow in number of vehicles but rapid in rate of increase followed by a gradual decrease in the rate of growth.

The number of persons per car in Ohio during the years 1913 to 1925 and the extension of the trend of persons per car to 1935 is shown in Figure 11. The estimated number of persons per car for each year is shown in Table 7.

Combining estimated population and estimated number of persons per car for each year, the predicted registration for each year is obtained. These estimates are also shown in Table 7.

TABLE 7.—Comparison of population and number of motor vehicles in the State of Ohio

Year	Registration (thousands)		Population ¹ (thousands)	Persons per car	
	Actual	Estimated		Actual	Estimated
1913	86	86	5,095	59.24	59.24
1914	123	127	5,197	42.60	40.92
1915	181	179	5,299	29.28	29.60
1916	252	245	5,402	21.44	22.05
1917	347	324	5,504	15.86	16.99
1918	413	416	5,606	13.57	13.48
1919	511	521	5,708	11.17	10.96
1920	621	637	5,810	9.36	9.12
1921	721	763	5,913	8.20	7.75
1922	859	897	6,015	7.00	6.71
1923	1,069	1,038	6,117	5.72	5.89
1924	1,242	1,181	6,219	5.01	5.27
1925	1,346	1,329	6,321	4.70	4.76
1926	1,480	1,475	6,424	—	4.36
1927	—	1,621	6,526	—	4.03
1928	—	1,763	6,628	—	3.76
1929	—	1,902	6,730	—	3.54
1930	—	2,035	6,833	—	3.36
1935	—	2,607	7,344	—	2.82

¹ Population as of July 1 of each year.

² Data not available when forecast was made. Estimate differs by 1.3 per cent from actual value in 1925, and by 0.3 per cent in 1926.

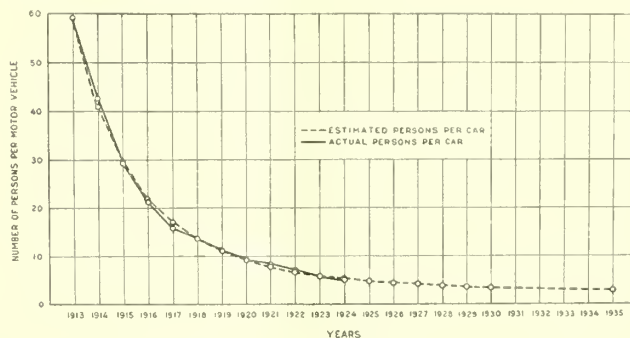


FIG. 11.—THE NUMBER OF PERSONS PER CAR IN OHIO FOR THE YEARS 1913 TO 1935 (BASED ON ESTIMATED POPULATION FOR INTERCENSAL YEARS)

On the basis of these predictions it is estimated that motor-vehicle registration in 1930 will be 2,035,000, with 3.36 persons per car, and in 1935, 2,607,000, with 2.82 persons per car. The increase in motor-vehicle registration between 1925 and 1930 is therefore expected to be 51 per cent and the increase between 1930 and 1935, 28 per cent.

The rate of population change varies greatly in different sections of the State, as does also the present number of persons per car.

The rates of decrease in persons per car within these areas, however, follow the same principle and are therefore in close agreement.

To allow for differences in the rate of population change and for differences in the present number of persons per car in the various sections of the State, the number of persons per car, based on estimated population and actual motor-vehicle registration in 1925, was obtained for each county of the State. To this 1925 value for each county was applied the rate of decrease in persons per car for the State between 1925 and 1930. The estimated registration for each county in 1930 was calculated by applying the estimated number of persons per car in 1930 for each county to the estimated 1930 population of that county. From the actual county registration in 1925 and the estimated registration in 1930 the percentage increase in regis-

(Continued on page 88)

N = the normal component of the resultant forces acting on a radial section of the arch ring.

$$n = \frac{E_s}{E_c} = 15.$$

P = a vertical concentrated load.

$p = \frac{N}{A}$, the average stress on a radial section.

r = the difference in elevation of the two supports. $r=0$ for a symmetrical arch.

s = the length of the arch axis.

ds = the length of one division of the arch axis.

T = the resultant thrust on a radial section of the arch.

t = the number of degrees of rise or fall of temperature.

u = the eccentricity of the normal thrust N .

V = the vertical component of the thrust T at any section.

V_o = the vertical reaction at the left support.

V_x = the vertical component of the thrust T at a point distant x from the left support.

x = the abscissa of any point on the arch axis. $x = z \frac{dx}{2}$.

dx = the horizontal projection of one division of the arch axis.

y = the ordinate of any point on the arch axis.

dy = the vertical projection of one division of the arch axis.

$z = \frac{2x}{dx}$ when the span is divided into a number of equal parts each equal to dx .

$$\Delta = \frac{ds}{I}$$

θ = the angle between the end tangents to the arch axis (fig. 3).

$\sum_a^l$ = the summation of all terms between the left support and the concentrated load distant a from the left support.

$\sum_a^r$ = the summation of all terms from the concentrated load to the right support.

$\sum_o^l$ = the summation of all terms from one support to the other.

$\Sigma = \sum_o^l$. When no limits are specified, it is understood that the summation is to be taken from one support to the other.

ϕ = the angle which any radial section makes with the vertical. (See fig. 3.)

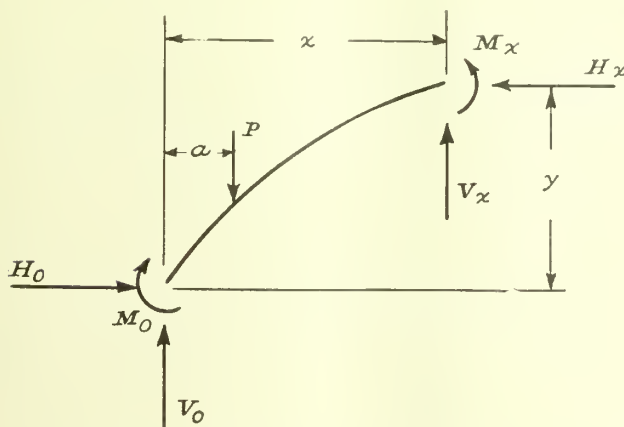


FIG. 2.—HINGELESS ARCH ACTED UPON BY LOAD P

DERIVATION OF FORMULAS

A hingeless arch is statically indeterminate, that is, the stresses in it can not be computed from the principles of statics alone. Let Figure 2 represent a portion of a hingeless arch acted upon by one or more loads P and held in equilibrium by the forces and moments

shown. There are six unknown quantities H_o , V_o , M_o , H_x , V_x , and M_x , but only three independent equations can be written from statics. For a single vertical load these equations are as follows:

$$H_x = H_o.$$

$$V_x = V_o - P.$$

$$M_x = M_o + V_o x - H_o y - P(x - a).$$

To determine the six unknown quantities, three additional equations must be supplied from some other source, and they may be derived by considering the elastic properties of the arch ring.

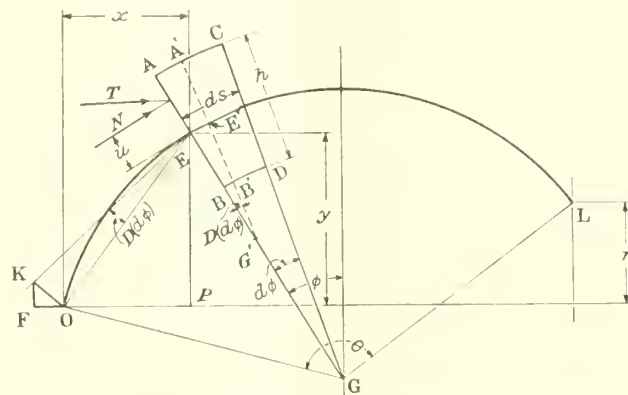


FIG. 3.—DIAGRAM FOR USE IN DERIVATION OF FORMULAS FOR STRESS IN AN ELASTIC ARCH

Referring to Figure 3, let ABCD represent a very small portion of the arch ring, cut out by radial planes AB and CD. Let the length of this small segment, measured along its center line, be ds and let the angle between the planes AB and CD be $d\phi$. Both ds and $d\phi$ are very small, and since the depth AB of the arch ring is small compared with the radius BG, the lengths AC and BD may be considered equal to ds without appreciable error in the results.

When the arch is subjected to loads or a change of temperature, there will be stress and deformation in the element ABCD. If we consider ABCD to be separated from the rest of the arch ring, the internal stresses in it must be held in equilibrium by external forces, so let us assume that T is the resultant of all the external forces acting on the plane AB and N the component of T perpendicular to AB. There is also a component of T parallel to AB, but it will cause only shear in the arch ring, which will be small and may be neglected. Neglecting the component parallel to AB is equivalent to assuming that N is the resultant, which we shall do. The forces acting on the plane AB are of course not concentrated in N , but are distributed over the plane, and we shall assume that they vary uniformly from the value at A to the value at B. Then if A is the area of the plane AB, the unit force or the unit internal stress acting at A will be $\left(\frac{N}{A} + \frac{Mh}{2I}\right)$ and at B, $\left(\frac{N}{A} - \frac{Mh}{2I}\right)$.

M , the bending moment is equal to Nu , u being the distance from the axis of the arch to the line of action of N . M is considered positive when u is positive, that is, when N acts above the axis of the arch, and negative when u is negative.

Consider first the elasticity of only the element ABCD and assume that the arch is fixed at the point L and free to move at point O and that the entire ring, with the exception of the element ABCD, is rigid. The plane CD will then be fixed and the plane AB will move until the internal stresses in the element ABCD are in equilibrium with the bending moment M and the thrust N . The plane AB will move to some position as A'B'. The unit stress in the extreme fiber AC will be $\left(\frac{N}{A} + \frac{Mh}{2I}\right)$, and since we can assume the length of this extreme fiber to be equal to the average length ds of the element, the distance AA' will be $-\left(\frac{N}{A} + \frac{Mh}{2I}\right)\frac{ds}{E}$. Likewise the unit stress in the fiber BD will be $\left(\frac{N}{A} - \frac{Mh}{2I}\right)$, and the distance BB' will be $-\left(\frac{N}{A} - \frac{Mh}{2I}\right)\frac{ds}{E}$, in which E is the modulus of elasticity of the material comprising the arch ring. AA' and BB' are both negative quantities because they represent decreases in the lengths AC and BD, respectively, and as it is assumed that N is positive, minus signs must be placed in front of the two quantities, as shown.

The movement of the plane AB may be considered as taking place in two separate stages: First, as moving along the axis of the arch ring from E to E', during which it remains parallel to its original position; and second, as rotating around the point E', through an angle $D(d\phi)$ and taking the position A'B'. The distance EE' which the plane AB moves along the axis of the arch ring is called $D(ds)$ because that is the change in the length of ds . Likewise, the angle through which it turns we call $D(d\phi)$ because that is the change in the angle $d\phi$.

When ds and $d\phi$ increase in magnitude, $D(ds)$ and $D(d\phi)$ will be considered as positive, and when ds and $d\phi$ decrease in magnitude $D(ds)$ and $D(d\phi)$ will be considered as negative. In the figure, the angle $d\phi$ is decreased by the deformation; therefore the angle $D(d\phi)$ is negative.

The angle $D(d\phi)$ is equal to $\frac{AA' - BB'}{h}$; and since AA' is equal to $-\left(\frac{N}{A} + \frac{Mh}{2I}\right)\frac{ds}{E}$, and BB' is equal to $-\left(\frac{N}{A} - \frac{Mh}{2I}\right)\frac{ds}{E}$, the angle $D(d\phi)$ is equal to

$$\left[-\left(\frac{N}{A} + \frac{Mh}{2I}\right)\frac{ds}{E} + \left(\frac{N}{A} - \frac{Mh}{2I}\right)\frac{ds}{E} \right] \frac{1}{h},$$

which reduces to $-\frac{Mds}{EI}$.

$$\text{Therefore } D(d\phi) = -M \frac{ds}{EI} \quad (1)$$

Now consider the effect on the point O of the rotating of the plane AB through the angle $D(d\phi)$. The arch ring is free to move at O. Since that part of the ring between O and E is rigid, when the plane AB turns through the angle $D(d\phi)$ the line EO will likewise describe the same angle. During this rotation the point O will move through an arc of a circle to K. OK will then be equal to OE times the angle $D(d\phi)$; and since the angle $D(d\phi)$ is negative, $OK = -OE D(d\phi)$. Since the angle $D(d\phi)$ is very small, we may consider that OK is a straight line and that the angle EOK is a right angle. From geometry, the angle FOK is equal to the angle PEO.

Then, $\frac{OF}{OK} = \frac{EP}{OE} = \frac{y}{OE}$. (OF represents an increase in span of arch and is therefore positive.)

$$OF = OK \frac{y}{OE} = -OE D(d\phi) \frac{y}{OE}$$

$$\text{Or, } OF = -D(d\phi)y$$

$$\text{But, } D(d\phi) = -M \frac{ds}{EI}$$

$$\text{Therefore, } OF = +My \frac{ds}{EI} \quad (2)$$

Also, $\frac{FK}{OK} = \frac{OP}{OE} = \frac{x}{OE}$. (FK represents an upward movement of the arch at point O and is therefore positive.)

$$FK = OK \frac{x}{OE} = -OE D(d\phi) \frac{x}{OE}$$

$$\text{Or, } FK = -D(d\phi)x$$

$$\text{By substitution, } FK = +Mx \frac{ds}{EI} \quad (3)$$

Now let us return and consider the effect on the point O of the other stage of the movement of the plane AB, that is, its movement along the axis of the arch during which it remains parallel to its original position. It is apparent that during this movement of the plane AB the change in position of the point O will correspond exactly with that of the point E; that is, point O will move a distance equal to $D(ds)$. Now

$D(ds)$ is equal to $\frac{1}{2}(AA' + BB')$, or to

$$\frac{1}{2} \left[-\left(\frac{N}{A} + \frac{Mh}{2I}\right)\frac{ds}{E} - \left(\frac{N}{A} - \frac{Mh}{2I}\right)\frac{ds}{E} \right],$$

which reduces to $-\frac{Nds}{AE}$.

$$\text{Therefore, } D(ds) = -\frac{Nds}{AE} \quad (4)$$

dx and dy are the horizontal and vertical projections of ds , and since ϕ is equal to the angle which ds makes with the horizontal, $dx = ds \cos \phi$, and $dy = ds \sin \phi$; also $D(dx) = D(ds) \cos \phi$ and $D(dy) = D(ds) \sin \phi$.

$$\text{By substitution, } D(dx) = -\frac{Nds}{AE} \cos \phi$$

$$\text{and } D(dy) = -\frac{Nds}{AE} \sin \phi$$

Substituting the values of $\cos \phi = \frac{dx}{ds}$, and $\sin \phi = \frac{dy}{ds}$,

$$D(dx) = -\frac{Ndx}{AE} \quad (5)$$

$$\text{and } D(dy) = -\frac{Ndy}{AE} \quad (6)$$

Equations 5 and 6 give the horizontal and vertical changes in position of the point O caused by the thrust N .

We now have the changes in position of the point O caused by both the bending moment M and the thrust N acting on the element ABCD when the point O is

Then:

$$\begin{aligned} M &= M_o + V_o x - H_o y, & \text{when } x < a \\ M &= M_o + V_o x - H_o y - (x-a), & \text{when } x > a \end{aligned} \quad (17)$$

$$N = H \cos \phi + V \sin \phi \quad (18)$$

in which, M_o , V_o , and H_o are, respectively, the bending moment, the vertical reaction, and the horizontal thrust at the left support.

In the following mathematical transformations, x and y are the variables, while a remains constant. Therefore, M_o , V_o , and H_o are constants because they depend on a and not on x or y . For that reason they may be taken outside of the summation signs.

Substituting equations 17 and 18 in equations 14, 15, and 16 we have:

$$\begin{aligned} & \sum_o^a [M_o + V_o x - H_o y] \Delta \\ & + \sum_o^l [M_o + V_o x - H_o y - (x-a)] \Delta = 0 \\ & \sum_o^a [M_o + V_o x - H_o y] y \Delta \\ & + \sum_o^l [M_o + V_o x - H_o y - (x-a)] y \Delta \\ & - \sum_o^l \left(\frac{H \cos \phi + V \sin \phi}{A} \right) dx + etl E = 0 \\ & \sum_o^a [M_o + V_o x - H_o y] x \Delta \\ & + \sum_o^l [M_o + V_o x - H_o y - (x-a)] x \Delta \\ & + \sum_o^l \left(\frac{H \cos \phi + V \sin \phi}{A} \right) dy - etr E = 0 \end{aligned}$$

These three equations may be written:

$$\begin{aligned} M_o \sum_o^l \Delta + V_o \sum_o^l x \Delta - H_o \sum_o^l y \Delta - \sum_o^l (x-a) \Delta &= 0 \\ M_o \sum_o^l y \Delta + V_o \sum_o^l xy \Delta - H_o \sum_o^l y^2 \Delta - \sum_o^l (x-a) y \Delta \\ &- \sum_o^l H \frac{\cos \phi}{A} dx - \sum_o^l V \frac{\sin \phi}{A} dx + etl E = 0 \\ M_o \sum_o^l x \Delta + V_o \sum_o^l x^2 \Delta - H_o \sum_o^l xy \Delta - \sum_o^l (x-a) x \Delta \\ &+ \sum_o^l H \frac{\cos \phi}{A} dy + \sum_o^l V \frac{\sin \phi}{A} dy - etr E = 0 \end{aligned}$$

In the last two equations H and V are still inside the summation sign, but $H = H_o$ and is therefore constant.

Also, $V = V_o$, for $x < a$
 $V = V_o - 1$, for $x > a$

Making this substitution in the three equations, we have:

$$M_o \sum_o^l \Delta + V_o \sum_o^l x \Delta - H_o \sum_o^l y \Delta - \sum_o^l (x-a) \Delta = 0$$

$$\begin{aligned} M_o \sum_o^l y \Delta + V_o \sum_o^l xy \Delta - H_o \sum_o^l y^2 \Delta - \sum_o^l (x-a) y \Delta \\ - H_o \sum_o^l \frac{\cos \phi}{A} dx - V_o \sum_o^a \frac{\sin \phi}{A} dx \\ - (V_o - 1) \sum_o^l \frac{\sin \phi}{A} dx + etl E = 0 \\ M_o \sum_o^l x \Delta + V_o \sum_o^l x^2 \Delta - H_o \sum_o^l xy \Delta - \sum_o^l (x-a) x \Delta \\ + H_o \sum_o^l \frac{\cos \phi}{A} dy + V_o \sum_o^a \frac{\sin \phi}{A} dy \\ + (V_o - 1) \sum_o^l \frac{\sin \phi}{A} dy - etr E = 0 \end{aligned}$$

These three equations may be written:

$$M_o \sum_o^l \Delta + V_o \sum_o^l x \Delta - H_o \sum_o^l y \Delta - \sum_o^l (x-a) \Delta = 0 \quad (19)$$

$$\begin{aligned} M_o \sum_o^l y \Delta + V_o \sum_o^l xy \Delta - H_o \sum_o^l y^2 \Delta - \sum_o^l (x-a) y \Delta \\ - H_o \sum_o^l \frac{\cos \phi}{A} dx - V_o \sum_o^l \frac{\sin \phi}{A} dx \\ + \sum_o^l \frac{\sin \phi}{A} dx + etl E = 0 \quad (20) \end{aligned}$$

$$\begin{aligned} M_o \sum_o^l x \Delta + V_o \sum_o^l x^2 \Delta - H_o \sum_o^l xy \Delta - \sum_o^l (x-a) x \Delta \\ + H_o \sum_o^l \frac{\cos \phi}{A} dy + V_o \sum_o^l \frac{\sin \phi}{A} dy \\ - \sum_o^l \frac{\sin \phi}{A} dy - etr E = 0 \quad (21) \end{aligned}$$

From equation 19

$$M_o = H_o \frac{\sum y \Delta}{\sum \Delta} - V_o \frac{\sum x \Delta}{\sum \Delta} + \frac{1}{\sum \Delta} \sum (x-a) \Delta$$

Substituting this value of M_o in equations 20 and 21 and collecting the terms containing H_o and V_o , we have:

$$\begin{aligned} V_o \left[- \sum y \Delta \frac{\sum x \Delta}{\sum \Delta} + \sum xy \Delta - \sum \frac{\sin \phi}{A} dx \right] \\ + H_o \left[\sum y \Delta \frac{\sum y \Delta}{\sum \Delta} - \sum y^2 \Delta - \sum \frac{\cos \phi}{A} dx \right] \\ = - \frac{\sum y \Delta}{\sum \Delta} \sum_o^l (x-a) \Delta + \sum_o^l (x-a) y \Delta \\ - \sum_o^l \frac{\sin \phi}{A} dx - etl E, \text{ and} \end{aligned}$$

$$\begin{aligned}
& V_o \left[-\frac{(\Sigma x \Delta)^2}{\Sigma \Delta} + \Sigma x^2 \Delta + \Sigma \frac{\sin \phi}{A} dy \right] \\
& + H_o \left[\Sigma x \Delta \frac{\Sigma y \Delta}{\Sigma \Delta} - \Sigma xy \Delta + \Sigma \frac{\cos \phi}{A} dy \right] \\
& = -\frac{\Sigma x \Delta}{\Sigma \Delta} \frac{l}{a} (x-a) \Delta + \frac{l}{a} (x-a) x \Delta \\
& + \frac{l}{a} \frac{\sin \phi}{A} dy + etr E = 0
\end{aligned}$$

These two equations may be written:

$$\begin{aligned}
& V_o \left[\Sigma x \Delta \left(y - \frac{\Sigma y \Delta}{\Sigma \Delta} \right) - \Sigma \frac{\sin \phi}{A} dx \right] \\
& - H_o \left[\Sigma y \Delta \left(y - \frac{\Sigma y \Delta}{\Sigma \Delta} \right) + \Sigma \frac{\cos \phi}{A} dx \right] \\
& = \frac{l}{a} (x-a) \Delta \left(y - \frac{\Sigma y \Delta}{\Sigma \Delta} \right) - \frac{l}{a} \frac{\sin \phi}{A} dx - etl E \dots \dots (22)
\end{aligned}$$

$$\begin{aligned}
& V_o \left[\Sigma x \Delta \left(x - \frac{\Sigma x \Delta}{\Sigma \Delta} \right) + \Sigma \frac{\sin \phi}{A} dy \right] \\
& - H_o \left[\Sigma x \Delta \left(y - \frac{\Sigma y \Delta}{\Sigma \Delta} \right) - \Sigma \frac{\cos \phi}{A} dy \right] \\
& = \frac{l}{a} (x-a) \Delta \left(x - \frac{\Sigma x \Delta}{\Sigma \Delta} \right) + \frac{l}{a} \frac{\sin \phi}{A} dy + etr E \dots \dots (23)
\end{aligned}$$

$$\text{Let } x = z \frac{dx}{2} \text{ and } a = k \frac{dx}{2}.$$

When the span is divided into 20 equal parts each of length dx , $l = 40 \frac{dx}{2}$. We can consider dx as a constant and z as a variable.

Making these substitutions, and dividing equation 22 by dx and equation 23 by $\frac{(dx)^2}{2}$ we have:

$$\begin{aligned}
& V_o \left[\frac{1}{2} \Sigma z \Delta \left(y - \frac{\Sigma y \Delta}{\Sigma \Delta} \right) - \Sigma \frac{\sin \phi}{A} \right] \\
& - H_o \left[\frac{1}{dx} \Sigma y \Delta \left(y - \frac{\Sigma y \Delta}{\Sigma \Delta} \right) + \Sigma \frac{\cos \phi}{A} \right] \\
& = \frac{1}{2} \frac{l}{a} (z-k) \Delta \left(y - \frac{\Sigma y \Delta}{\Sigma \Delta} \right) - \frac{l}{a} \frac{\sin \phi}{A} - 20 et E \dots \dots (24)
\end{aligned}$$

$$\begin{aligned}
& V_o \left[\frac{1}{2} \Sigma z \Delta \left(z - \frac{\Sigma z \Delta}{\Sigma \Delta} \right) + \frac{2}{(dx)^2} \Sigma \frac{\sin \phi}{A} dy \right] \\
& - H_o \left[\frac{1}{dx} \Sigma z \Delta \left(y - \frac{\Sigma y \Delta}{\Sigma \Delta} \right) - \frac{2}{(dx)^2} \Sigma \frac{\cos \phi}{A} dy \right] \\
& = \frac{1}{2} \frac{l}{a} (z-k) \Delta \left(z - \frac{\Sigma z \Delta}{\Sigma \Delta} \right) + \frac{2}{(dx)^2} \frac{l}{a} \frac{\sin \phi}{A} dy \\
& + \frac{2}{(dx)^2} etr E \dots \dots (25)
\end{aligned}$$

In the above equations the terms containing $\sin \phi$ and $\cos \phi$ are those which include the effect of the axial or direct stress. To neglect them would be to neglect the axial stress which is sometimes done because it is usually a small part of the total stress. It may not be desirable to neglect this stress entirely, but we can shorten the work considerably by neglecting some of the terms.

The terms $\Sigma \frac{\sin \phi}{A}$, $\frac{2}{(dx)^2} \Sigma \frac{\sin \phi}{A} dy$, $\frac{2}{(dx)^2} \Sigma \frac{\cos \phi}{A} dy$, $\frac{l}{a} \frac{\sin \phi}{A}$, and $\frac{2}{(dx)^2} \frac{l}{a} \frac{\sin \phi}{A} dy$ are all very small compared with the terms to which they are added, and probably no accuracy will be sacrificed by neglecting them. The data upon which all of the terms depend are obtained by scaling a large-scale drawing, and this drawing, while probably more accurate in dimensions than the dimensions of the arch as built is not sufficiently accurate to make the above-mentioned terms significant.

Omitting the above-mentioned terms, we have:

$$\begin{aligned}
& V_o \frac{1}{2} \Sigma z \Delta \left(y - \frac{\Sigma y \Delta}{\Sigma \Delta} \right) - H_o \left[\frac{1}{dx} \Sigma y \Delta \left(y - \frac{\Sigma y \Delta}{\Sigma \Delta} \right) + \Sigma \frac{\cos \phi}{A} \right] \\
& = \frac{1}{2} \frac{l}{a} (z-k) \Delta \left(y - \frac{\Sigma y \Delta}{\Sigma \Delta} \right) - 20 et E \dots \dots (26)
\end{aligned}$$

$$\begin{aligned}
& V_o \frac{1}{2} \Sigma z \Delta \left(z - \frac{\Sigma z \Delta}{\Sigma \Delta} \right) - H_o \frac{1}{dx} \Sigma z \Delta \left(y - \frac{\Sigma y \Delta}{\Sigma \Delta} \right) \\
& = \frac{1}{2} \frac{l}{a} (z-k) \Delta \left(z - \frac{\Sigma z \Delta}{\Sigma \Delta} \right) + \frac{2}{(dx)^2} etr E \dots \dots (27)
\end{aligned}$$

In these two equations the coefficients of H_o and V_o contain no summations except those for the entire arch ring. For that reason these coefficients are independent of the loading and may be considered as constants.

Therefore we may let:

$$B = \frac{1}{2} \Sigma z \Delta \left(y - \frac{\Sigma y \Delta}{\Sigma \Delta} \right) \dots \dots (28)$$

$$C = \frac{1}{dx} \Sigma y \Delta \left(y - \frac{\Sigma y \Delta}{\Sigma \Delta} \right) + \Sigma \frac{\cos \phi}{A} \dots \dots (29)$$

$$F = \frac{1}{2} \Sigma z \Delta \left(z - \frac{\Sigma z \Delta}{\Sigma \Delta} \right) \dots \dots (30)$$

$$G = \frac{1}{dx} \Sigma z \Delta \left(y - \frac{\Sigma y \Delta}{\Sigma \Delta} \right) \dots \dots (31)$$

Then the two equations may be written:

$$V_o B - H_o C = \frac{1}{2} \frac{l}{a} (z-k) \Delta \left(y - \frac{\Sigma y \Delta}{\Sigma \Delta} \right) - 20 et E$$

$$V_o F - H_o G = \frac{1}{2} \frac{l}{a} (z-k) \Delta \left(z - \frac{\Sigma z \Delta}{\Sigma \Delta} \right) + \frac{2}{(dx)^2} etr E$$

From the first of these two equations:

$$H_o = V_o \frac{B}{C} - \frac{1}{C} \left[\frac{1}{2} \frac{l}{a} (z-k) \Delta \left(y - \frac{\Sigma y \Delta}{\Sigma \Delta} \right) - 20 et E \right] \dots (32)$$

and by substitution:

$$V_o = \frac{\frac{1}{2} \frac{\Sigma}{a} (z-k) \Delta \left(z - \frac{\Sigma z \Delta}{\Sigma \Delta} \right) + \frac{2}{(dx)^2} etr E}{F - \frac{BG}{C}}$$

$$\frac{G \left[\frac{1}{2} \frac{\Sigma}{a} (z-k) \Delta \left(y - \frac{\Sigma y \Delta}{\Sigma \Delta} \right) - 20 et E \right]}{F - \frac{BG}{C}} \quad (33)$$

It is more convenient to consider the effect of the loads and temperature separately. When the effect of the loads is considered without a change of temperature, t in the formulas becomes zero. Then the effect of a change of temperature may be computed by making the terms which contain the effect of the loads equal to zero.

Making this separation and collecting the formulas which are used in computing the stresses in an arch ring, we have:

$$V_o = \frac{\frac{1}{2} \frac{\Sigma}{a} (z-k) \Delta \left(z - \frac{\Sigma z \Delta}{\Sigma \Delta} \right)}{F - \frac{BG}{C}}$$

$$\frac{G \frac{1}{2} \frac{\Sigma}{a} (z-k) \Delta \left(y - \frac{\Sigma y \Delta}{\Sigma \Delta} \right)}{F - \frac{BG}{C}} \quad (34)$$

$$H_o = \frac{V_o B - \frac{1}{2} \frac{\Sigma}{a} (z-k) \Delta \left(y - \frac{\Sigma y \Delta}{\Sigma \Delta} \right)}{C} \quad (35)$$

$$M_o = \frac{dx}{\Sigma \Delta} \frac{1}{2} \frac{\Sigma}{a} (z-k) \Delta + H_o \frac{\Sigma y \Delta}{\Sigma \Delta} - V_o \frac{dx}{2} \frac{\Sigma z \Delta}{\Sigma \Delta} \quad (36)$$

$$V_x = \begin{cases} V_o, & \text{for } x < a \\ V_o - 1, & \text{for } x > a \end{cases} \quad (37)$$

$$H_x = H_o \quad (38)$$

$$M_x = \begin{cases} M_o + V_o z \frac{dx}{2} - H_o y, & \text{for } x < a \\ M_o + [V_o z - (z-k)] \frac{dx}{2} - H_o y, & \text{for } x > a \end{cases} \quad (39)$$

$$N_x = H_o \cos \phi + V_x \sin \phi \quad (40)$$

$$V_t = \frac{2r}{(dx)^2} + 20 \frac{G}{C} et E$$

$$F - \frac{BG}{C} \quad (41)$$

$$H_t = \frac{V_t B + 20 et E}{C} \quad (42)$$

$$M_t = -H_t \left(y - \frac{\Sigma y \Delta}{\Sigma \Delta} \right) + V_t \frac{dx}{2} \left(z - \frac{\Sigma z \Delta}{\Sigma \Delta} \right) \quad (43)$$

$$f_c = \frac{N}{A} \pm M \frac{h}{2I} \quad (44)$$

Equations 34 to 44, inclusive, are applicable to all concrete arches with fixed ends, whether symmetrical or unsymmetrical. However, if the arch is symmetrical, some of the terms in the above equations will become zero and the equations will be simplified.

If the arch is symmetrical, the values of Δ will be symmetrical—that is, the value of Δ for point 1' will be the same as for point 1 and for point 2' the same as for point 2, etc. The value of z for points 1 and 1' combined is $(1+39)=40$ and for points 2 and 2' combined it is $(3+37)=40$ and in the same way for all pairs of symmetrical points combined the value of z is always 40. Therefore, for symmetrical arches $\Sigma z \Delta = 20 \Sigma \Delta$. The values of y are also symmetrical and we have $\Sigma y \Delta = 20 \Sigma y \Delta$.

These values transform equation 28 as follows:

$$B = \frac{1}{2} \Sigma z \Delta \left(y - \frac{\Sigma y \Delta}{\Sigma \Delta} \right) = \frac{1}{2} \Sigma z y \Delta - \frac{1}{2} \Sigma z \Delta \frac{\Sigma y \Delta}{\Sigma \Delta}$$

$$= \frac{1}{2} \times 20 \Sigma y \Delta - \frac{1}{2} \times 20 \Sigma \Delta \frac{\Sigma y \Delta}{\Sigma \Delta}$$

$$= 10 \Sigma y \Delta - 10 \Sigma y \Delta = 0$$

Therefore, for symmetrical arches

$$B = 0$$

$$F = \frac{1}{2} \Sigma z \Delta \left(z - \frac{\Sigma z \Delta}{\Sigma \Delta} \right) = \frac{1}{2} \Sigma z^2 \Delta - \frac{1}{2} \Sigma z \Delta \frac{\Sigma z \Delta}{\Sigma \Delta}$$

$$= \frac{1}{2} \Sigma z^2 \Delta - \frac{1}{2} \times 20 \Sigma \Delta \frac{20 \Sigma \Delta}{\Sigma \Delta}$$

$$= \frac{1}{2} \Sigma z^2 \Delta - 200 \Sigma \Delta$$

$G = 0$ for the same reason as given for B

Therefore, for symmetrical arches we have the following formulas:

FORMULAS FOR SYMMETRICAL ARCHES

$$C = \frac{1}{dx} \Sigma y \Delta \left(y - \frac{\Sigma y \Delta}{\Sigma \Delta} \right) + \Sigma \frac{\cos \phi}{A}$$

$$F = \frac{1}{2} \Sigma z^2 \Delta - 200 \Sigma \Delta$$

$$V_o = \frac{\frac{1}{2} \frac{\Sigma}{a} (z-k) (z-20) \Delta}{F} \quad (45)$$

$$H_o = \frac{-\frac{1}{2} \frac{\Sigma}{a} (z-k) \Delta \left(y - \frac{\Sigma y \Delta}{\Sigma \Delta} \right)}{C} \quad (46)$$

$$M_o = \frac{dx}{\Sigma \Delta} \frac{1}{2} \frac{\Sigma}{a} (z-k) \Delta + H_o \frac{\Sigma y \Delta}{\Sigma \Delta} - 20 \frac{dx}{2} V_o \quad (47)$$

$$V_x = \begin{cases} V_o, & \text{when } x < a \\ V_o - 1, & \text{for a single unit load when } x > a \end{cases} \quad (48)$$

$$H_x = H_o$$

$$M_x = M_o + \left[V_o z - (z^2 - k) \right] \frac{dx}{2} - H_o y \quad (49)$$

$$N_x = H \cos \phi + V_x \sin \phi \quad (50)$$

$$V_t = 0 \quad (51)$$

$$H_t = 20 \frac{etE}{C} \quad (52)$$

$$M_t = -H_t \left(y - \frac{\sum y \Delta}{\sum \Delta} \right) \quad (53)$$

$$f_c = \frac{N}{A} \pm M \frac{h}{2I} \quad (54)$$

The term $\frac{\cos \phi}{A}$ in some of the equations includes the effect of the so-called axial or rib shortening stress. This stress is sometimes considered separately from the other stresses and sometimes is neglected entirely. If the rise of the arch is relatively large, the axial stress is small and little accuracy is sacrificed by neglecting it, but if the arch is flat its effect is considerable and it should not be neglected.

Since it is easily taken care of on the forms, there appears to be no good reason for neglecting it or separating it from the rest of the stress, so it will be left in the formula and on the forms in its correct place, and the axial stress will be included in all cases.

CALCULATIONS FOR A SYMMETRICAL ARCH

The method of making the calculations for a symmetrical arch can best be explained by working out an actual example, and each step will be taken up in order and explained in detail. All calculations are entered in Tables 1 to 9, forms for which can be provided on five sheets of letter-size paper, and these forms are grouped in the text as used by the authors. Numerical values and plus and minus signs, which are the same for all arches, are shown in boldface type. In actual practice, blank forms made by the white-print process

and showing column headings and other information common to all symmetrical arches are used. The formulas to be used are those developed for symmetrical arches. The formulas and forms for unsymmetrical arches could be used, but most arches are symmetrical, and the work is much reduced by using the simplified rather than the general formulas.

Assume that a 60-foot arch is to be designed for a uniform live load of 125 pounds per square foot (concentrated loads may be used without difficulty) and that the arch must withstand temperature stresses caused by a rise in temperature of 30° F. or a drop of 40° F. The tentative proportions of the arch and its reinforcement are determined and half of the arch ring plotted on detail paper on a scale of 1 inch to 2 feet. The rather large scale is used because the stresses to be determined will depend on scaled dimensions from this drawing. A similar drawing on a smaller scale is made on letter-size paper to accompany the tables of calculations as illustrated in Figure 5 (sheet 1 of forms).

After plotting one-half of the arch ring, draw the axis of the arch ring, which is a curve lying halfway between the intrados and extrados. Draw a vertical line through the springing line of the intrados until it intersects the arch axis as shown in Figure 5. This point is the origin of coordinates and is referred to as the origin, or point *O*. Through the origin draw a horizontal line, and on this line divide the half span into 10 equal parts. The length of each part will be equal to *dx*. On sheet 2 (Tables 1, 2, and 3) of the forms record in the places indicated the values of *l*, *dx* and the rise, which is the vertical distance from the arch axis at the crown to the horizontal line drawn through the origin. At the center of each *dx* erect a perpendicular to intersect with the arch axis, and mark these intersections 1, 2, 3, etc. In working out this example maximum stresses will be determined at points 0, 3, 8, and 10½ and the abutment pressure on the foundation determined.

The general method of procedure is to place a load of unity successively at each of the points on the arch ring and compute coefficients which can be applied to the actual dead and live loads for determining moments, shears, and thrusts.

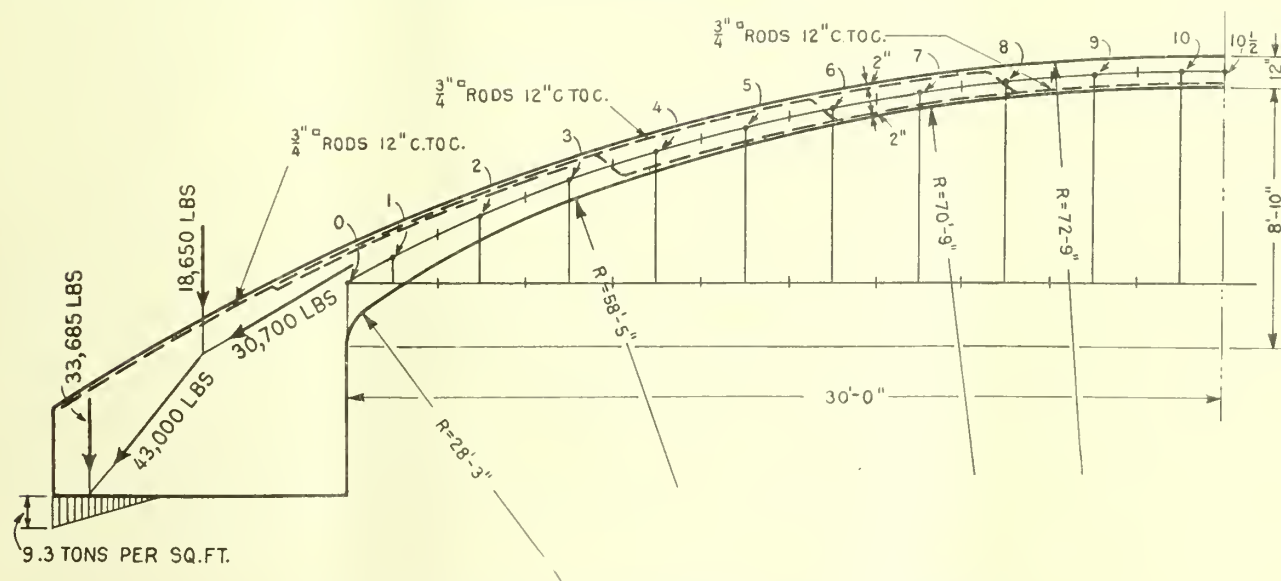


FIG. 5.—SKETCH OF ARCH TO ACCOMPANY CALCULATION SHEETS

TABLE 1.—Computations for Δ

1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18
Points	h	h^2	$I_c = \frac{h^3}{12}$	h	$\frac{h}{2} - d'$	$\left(\frac{h}{2} - d'\right)^2$	$I_s = \left(\frac{h}{2} - d'\right)^2 n A_s$	$I = I_c + I_s$	ds	$\Delta = \frac{ds}{I}$	$z - 20$	$(z - 20)\Delta$	$\frac{1}{\Sigma} \Sigma Q = \frac{1}{\Sigma} \Sigma (z - 20)\Delta$	$\frac{1}{\Sigma} \Sigma (z - k) Q$	z^2	$z^2 \Delta$	V_o
0	2.30	12.167	1.014	1.15	0.98	0.960	0.112	1.126									
1	1.98	7.762	.647	.99	.82	.672	.079	.726	3.43	4.7	19	89.3	89.3	20,091.6	1,522	7,153.4	1.00000
2	1.51	3.443	.287	.75	.58	.336	.039	.326	3.29	10.1	17	171.7	261.0	20,062.3	1,378	13,917.8	.99556
3	1.30	2.197	.183	.65	.48	.230	.013	.196	3.17	16.2	15	243.0	504.0	19,741.3	1,250	20,250.0	.98256
4	1.18	1.643	.137	.59	.42	.176	.010	.147	3.13	21.3	13	276.9	780.9	19,237.3	1,138	24,239.4	.95748
5	1.10	1.331	.111	.55	.38	.144	.008	.119	3.09	26.0	11	286.0	1,066.9	18,456.4	1,042	27,092.0	.91861
6	1.05	1.158	.096	.52	.35	.122	.007	.103	3.05	29.6	9	266.4	1,333.3	17,389.5	962	28,475.2	.86551
7	1.02	1.061	.088	.51	.34	.116	.007	.095	3.02	31.8	7	222.6	1,555.9	16,056.2	898	28,556.4	.79915
8	1.02	1.061	.088	.51	.34	.116	.007	.095	3.01	31.7	5	158.5	1,714.4	14,500.3	850	26,945.0	.72171
9	1.00	1.000	.083	.50	.33	.109	.006	.089	3.00	33.7	3	101.1	1,815.5	12,785.9	818	27,566.4	.63638
10	1.00	1.000	.083	.50	.33	.109	.006	.089	3.00	33.7	1	33.7	1,849.2	10,970.4	802	27,027.4	.54602
											$\frac{1}{2} \Sigma \Delta =$	238.8			$\Sigma z^2 \Delta =$	231,223.2	

TABLE 3.—Computations for H_o

19	20	21	22	23	24	25	26	27	28	29	30	$\text{Rise} = 1.22 \text{ ft.}$ $dx = \frac{l}{20} = 3.00$ $\frac{1}{2} dx = 1.50$ $\frac{\Sigma y \Delta}{\Sigma \Delta} = 5.85585$ $F = \frac{1}{2} \Sigma z^2 \Delta - 200 \Sigma \Delta = 20,091.6$ $C = \frac{1}{dx} \frac{\Sigma y \Delta}{\Sigma \Delta} \left(y - \frac{\Sigma y \Delta}{\Sigma \Delta} \right)$ $+\Sigma \frac{\cos. \phi}{A} = 375.409$			
Points	y	Δ	$y \Delta$	$y - \frac{\Sigma y \Delta}{\Sigma \Delta}$	$\Delta \left(y - \frac{\Sigma y \Delta}{\Sigma \Delta} \right)$	$\frac{1}{\Sigma \Delta} \left(\frac{\Sigma Q}{y - \frac{\Sigma y \Delta}{\Sigma \Delta}} \right)$	$\frac{1}{\Sigma \Delta} (z - k) Q$	$y \Delta \left(y - \frac{\Sigma y \Delta}{\Sigma \Delta} \right)$	$\cos \phi$	$\frac{\cos \phi}{A}$	H_o				
0	0								0.838						
1	0.88	4.7	4.136	-4.976	-23.387	23.352	0.000	20.581	.874	0.441	0.000				
2	2.32	10.1	23.432	-3.536	-35.714	59.066	23.352	82.856	.911	.603	.062				
3	3.51	16.2	56.862	-2.346	-38.005	97.071	82.418	133.398	.946	.728	.219				
4	4.48	21.3	95.426	-1.376	-29.309	126.380	179.489	131.304	.948	.804	.478				
5	5.29	26.0	137.540	-0.566	-14.716	141.096	305.869	77.848	.971	.883	.815				
6	5.93	29.6	175.528	+0.074	2.190	138.906	446.965	12.987	.984	.938	1.191				
7	6.48	31.8	206.064	+0.624	19.843	119.063	585.871	128.583	.993	.974	1.561				
8	6.85	31.7	217.145	+0.994	31.510	87.553	704.934	215.844	.996	.976	1.878				
9	7.10	33.7	239.270	+1.244	41.923	45.630	792.487	297.653	1.000	1.000	2.111				
10	7.21	33.7	242.977	+1.354	45.630	0.000	838.117	328.992	1.000	1.000	2.232				
$\frac{1}{2} \Sigma y \Delta =$											1,398.378			$\frac{1}{2} \Sigma =$	8.347

Span = $l = 60.00$ ft.

Rise = 7.22 ft.

$$dx = \frac{l}{20} = 3.00$$

$$\frac{1}{2} dx = 1.50$$

$$\frac{\Sigma y\Delta}{\Sigma \Delta} = 5.85585$$

$$F = \frac{1}{2} \Sigma z^2 \Delta - 200 \Sigma \Delta = 20,091.6$$

$$C = \frac{1}{dx} \frac{1}{\Sigma} \Sigma y\Delta \left(y - \frac{\Sigma y\Delta}{\Sigma \Delta} \right) + \frac{\cos \phi}{A} = 375.409$$

$$V_o = \frac{\frac{1}{2} \Sigma (z - k) (z - 20) \Delta}{F} \quad H_o = \frac{-\frac{1}{2} \Sigma (z - k) \Delta \left(y - \frac{\Sigma y\Delta}{\Sigma \Delta} \right)}{C}$$

$$H_t = \frac{20}{C} \frac{dE}{C} = \frac{34560}{C} t = \frac{+2,762}{-3,682} \text{ for } t = \begin{matrix} +30 \\ -40 \end{matrix}$$

$$\text{Col. 15} = \frac{1}{2} \Sigma (z - k) (z - 20) \Delta$$

$$\text{Col. 26} = \frac{1}{2} \Sigma (z - k) \Delta \left(y - \frac{\Sigma y\Delta}{\Sigma \Delta} \right)$$

Table 1.—Computations for Δ .—Scale accurately the thickness of the arch ring h at each point and set down the result in column 2. Columns 3 to 9, inclusive, are used in finding the moments of inertia of the sections of the arch ring at the various points. $I = I_c + I_s$, in which I is the total moment of inertia of the section about the axis of the arch, I_c the moment of inertia of the concrete, and I_s the moment of inertia of the steel. It is convenient to consider a strip of the arch ring 1 foot in width and scale all dimensions in feet, then $I_c = \frac{bh^3}{12}$,

or, since $b = 1$, $I_c = \frac{h^3}{12}$, which is tabulated in column 4.

$I_s = I_o + A_s j^2$ in which I_s is the moment of inertia of the steel about the axis of the arch, I_o is the moment of inertia of the steel about its own axis, A_s is the area of the steel, and j is the distance from the axis of the arch to the axis of the steel. Since the strength of steel is equivalent to n times that of the same area of concrete, and all of the dimensions are in feet, A_s in the above formula must be taken as equal to n times the total area of steel in square inches divided by 144.

$$n = \frac{E_s}{E_c} = 15 \quad j = \frac{h}{2} - d'$$

in which d' is the distance from the surface of the con-

crete to the center of the steel. I_o is so small that it may be neglected.

Having computed I for each section and having the results tabulated in column 9, next scale ds for each section along the axis of the arch ring. The values of Δ are then computed. This should be carefully done because there is no check on the work and the accuracy of all of the rest of the columns will depend on the accuracy of column 11.

Special method of computation for terms involving

$\frac{1}{2} \Sigma (z - k)$.—Column 12 (also 16 and 32) is the same for all arches and is already filled out on the printed form. In preparing the forms the values of z for the different points have been determined from the formula $x = z \frac{dx}{2}$. By inspection of Figure 1 it is evident that the value of z for point 1 is 1, for point 2 it is 3, for point 10 it is 19, etc.

The numerators of the formulas for V_o and H_o and one term of the formula for M_o contain a term which may be written in the form,

$$\frac{1}{2} \Sigma (z - k) Q$$

In each of these formulas Q has a different value, but the method of procedure in arriving at the value of the above expression is the same in each case and the term Q is used in order that a single explanation may serve for three cases.

In the formula for V_o , $Q = (z-20)\Delta$

In the formula for H_o , $Q = \Delta \left(y - \frac{\sum y\Delta}{\sum \Delta} \right)$

In the formula for M_o , $Q = \Delta$.

In each case Q depends solely on the properties of the arch and has the values Q_1 , Q_2 , etc., for the points along the arch ring.

We will consider first the formula for M_o because we have there the values of $Q = \Delta$ for the entire arch ring while in the other two cases we have the values of Q for only one-half of the arch ring. Column 35 of Table 4 is copied from column 11 of Table 1. Table 4, A, has been inserted to show the manner in which the values in columns 36 and 37 are secured. This table and Table 4, B, are not part of the forms and are inserted for purposes of explanation only. Column 36 of Table 4, A, shows that $\sum Q$ for any point is the sum of values of Q for all points to the right of the point considered and is a matter of successive addition starting at the bottom of the column.

To understand the derivation of column 37 in Tables 4 and 4, A, it is necessary to study Table 4, B. In this table it is considered that a unit load is successively placed at each point and in each case the value of $\frac{1}{2} \sum_a^l (z-k)Q$ determined for each point to the right of the load. Values are not determined to the left of the load since the formula requires the values only where z is greater than k . At the point of load application z is equal to k and the expression becomes zero. The determinations of z and k for Table 4, B, are obvious from an examination of Figure 1. The value of $\frac{1}{2} \sum_a^l (z-k)Q$ for a unit load at any point is the sum of the column headed $\frac{1}{2} (z-k)Q$ and bearing the proper point designation. An inspection will show that the totals of these columns are the same as the quantities obtained in column 37 of Table 4, A, by the simple process of successive addition.

Table 2.—Computations for V_o .—We can now return to Table 2 of the forms and compute the values of V_o from the formula

$$V_o = \frac{\frac{1}{2} \sum_a^l (z-k)(z-20)\Delta}{\frac{1}{2} \sum z^2 \Delta - 200 \sum \Delta}$$

In this formula Q , as used in the preceding explanation is equal to $(z-20)\Delta$. The values of $(z-20)$ are the same for all arches when the arch ring is divided into 20 parts so they are permanently printed in column 12 of the forms. The values of $(z-20)\Delta$ are computed by multiplying each term in column 11 by the corresponding term in column 12 and the results written in column 13. If Table 2 were made out for the entire arch ring the computation of columns 14 and 15 would be exactly as explained for columns 36 and 37 in Table 4, A, but due to the symmetry of the arch ring it is necessary to use only the points on one side of the crown.

The values of Δ are symmetrical. That is the value of Δ_{20} is the same as Δ_1 , etc. The values of $(z-20)$ on the right-hand side of the arch are numerically the

same as those on the left but with the opposite algebraic sign. Therefore the values of $Q = (z-20)\Delta$ on the right-hand half of the arch are the same numerically as those on the left but of opposite algebraic sign. Referring to Table 4, A, we see that the quantity in column 36 for point 10 is $(Q_{20} + Q_{19} + \dots + Q_{11})$ which is equal to $-(Q_1 + Q_2 + \dots + Q_{10})$. Therefore we find the sum of column 13 and write it opposite point 10 in column 14 but with opposite algebraic sign. We then continue the process of successive addition to the top of the column as previously explained.



A FILLED-SPANDREL ARCH OF THE TYPE ILLUSTRATED BY THE EXAMPLE

Referring to Table 4, A, we see that the quantity in column 37 opposite point 10 which corresponds to the bottom figure in column 15 of Table 2 is $(10Q_{20} + 9Q_{19} + \dots + Q_{11})$ and this is equal to the sum of column 14. Therefore we find the sum of column 14 and write it in the bottom space of column 15. Next we add the figure in column 14 opposite point 9 and set down the result in column 15 opposite point 9 and continue the process of successive addition to the top of the column. Examination of column 37 of Table 4, A, and column 14 of Table 2 will demonstrate the correctness of this procedure. As a partial check on the numerical work it should be noted that the top figure of column 14 is numerically the same as the top figure of column 13. Column 15 now contains the values of $\frac{1}{2} \sum (z-k)(z-20)\Delta$ for a unit load placed successively at each point on the left half of the arch ring.

The denominator of the equation for V_o ; that is, $F = \frac{1}{2} \sum z^2 \Delta - 200 \sum \Delta$, is independent of the loading and is easily computed. The values of z^2 are permanently printed in column 16 of the forms and for convenience the symmetrical values are combined; that is, for points 1 and 20, $z^2 = (1^2 + 39^2) = 1522$. For points 2 and 19 combined, $z^2 = (3^2 + 37^2) = 1378$, etc.

Column 17 is computed by multiplying each term in column 16 by the corresponding values of Δ in column 11. The sum of column 17 is equal to $\sum z^2 \Delta$ and the sum of column 11 is equal to $\frac{1}{2} \sum \Delta$. The value of F is then computed and entered on the form. As a check on the numerical work it should be noted that the top figure of column 15 is equal to the value of F .

V_o can now be computed and tabulated in column 18 by dividing each term in column 15 by F or, since it is easier to multiply than to divide on most calculating machines, it will be quicker to find the reciprocal of F and multiply.

Table 3.—Computations for H_o .—Table 3 is used in computing H_o from the formula

$$H_o = \frac{-\frac{1}{2} \sum \left(z - k \right) \Delta \left(y - \frac{\sum y \Delta}{\sum \Delta} \right)}{C}$$

In this formula the denominator C is equal to

$$\frac{1}{dx} \sum y \Delta \left(y - \frac{\sum y \Delta}{\sum \Delta} \right) + \frac{1}{\sum} \frac{\cos \phi}{A}$$

and is independent of the loading. The numerator depends on the position of the unit load and is computed in a way similar to that used for the formula for V_o . Values of y are scaled from the drawing and tabulated in column 20. For convenience the values of Δ are copied in column 21. The values of $y\Delta$ are found by multiplying, and then the value of $\frac{\sum y \Delta}{\sum \Delta}$ may be found by dividing the sum of column 22 by the sum of column 11. Column 23 is computed by subtracting the constant value of $\frac{\sum y \Delta}{\sum \Delta}$ from each value of y . Then column 24 may be computed by multiplying each term in column 23 by the corresponding value of Δ . The sum of column 24 should be zero. If it is not it is due to some inaccuracy which should be found before going further. The error will be in column 21, 22, 23, or 24. Of course allowance should be made for the fact that decimal places were dropped in the value of $\frac{\sum y \Delta}{\sum \Delta}$ and that inaccuracy is carried into columns 24 and 25. For that reason the sum of column 24 will usually not check exactly, but it is not difficult to determine if the inaccuracy is from that cause or a mistake.

Columns 25 and 26 are computed from column 24 in exactly the same manner as columns 14 and 15 were computed from column 13.

The bottom figure in column 25 is equal to the sum of the figures of column 24 for the right-hand half of the arch, which are not shown, but they are the same as the figures which are shown and their sum is also zero. To this zero we add the last figure shown in column 24 and write the result in the next space above in column 25 and continue the process of successive addition to the top of the column. The top figure of column 25 should be approximately equal numerically to the top figure of column 24.

The bottom figure in column 26 is the sum of the figures in the half of column 25 which is not shown plus the figure opposite point 10, but since the latter figure is always zero it makes no difference in the sum. This sum is equal to minus the sum of the figures shown. This may be understood from consideration of Table 4, A. Thus we find the sum of the figures shown in column 25 and write it in the bottom space of column 26. Then we add to it the figure opposite point 9 in column 25 and continue the process of successive addition to the top of the column, observing the correct algebraic sign. Then all of the figures in column 26 will be negative and the top figure will be zero.

The denominator of the equation for H_o that is

$$C = \frac{1}{dx} \sum y \Delta \left(y - \frac{\sum y \Delta}{\sum \Delta} \right) + \sum \frac{\cos \phi}{A}$$

is independent of the loading and is computed in columns 27, 28, and 29 as indicated by the headings of the columns. $\cos \phi$ is tabulated in column 28. It might

be computed from the formula $\cos \phi = \frac{dx}{ds}$, but since $\cos \phi$ varies so little for small angles it will be found more accurate to compute $\sin \phi$ from dimensions scaled on the drawing and then find the corresponding values of $\cos \phi$ in a table of trigonometric functions.

Column 29 is computed by dividing each term in column 28 by the area A . Since a strip of the arch ring 1 foot wide is being considered, A is equal to h plus n times the area of steel in square feet. Since the area of steel in square feet is small and the effect of $\cos \phi$ is relatively small, it is sufficiently accurate to assume, for this purpose, that A is equal to h . C , the denominator of the equation for H_o , is found by dividing the sum of column 27 by $\frac{1}{2} dx$ and adding twice the sum of column 29. The value of C should be set down in the space provided on the same sheet with Table 3. H_o may now be computed for a load of unity at each point by dividing each term of column 26 by C and the results are tabulated in column 30. Note that the algebraic sign is changed because the formula for H_o is preceded by a minus sign.

Table 4.—Computation for M_o .— M_o is computed in Table 4 from the formula

$$M_o = H_o \frac{\sum y \Delta}{\sum \Delta} - 20 \frac{dx}{2} V_o + \frac{dx}{\sum \Delta} \times \frac{1}{2} \sum_a (z - k) \Delta$$

The values of z are permanently printed in column 32. The values of H_o , V_o and Δ have been previously found and for convenience are copied in columns 33, 34, and 35. The values of H_o and Δ are symmetrical so the values found for the left-hand half of the arch ring may be copied for the right-hand half. The values of V_o for the right half of the arch may be found by subtracting each value for the left half from unity.

Column 36 is computed by summing up column 35, starting with zero in the bottom space for point 1'. Column 37 is computed by summing up column 36 in the same way starting with zero at the bottom opposite point 1' as previously explained. Point 1' for symmetrical arches is the same as point 20 used in the explanation. Column 38 is computed by multiplying

each term in column 37 by $\frac{dx}{\sum \Delta}$. Column 39 is computed

by multiplying each value of H_o by $\frac{\sum y \Delta}{\sum \Delta}$. Column 40 is computed by multiplying each value of V_o by $-20 \frac{dx}{2}$.

The sum of column 39 should equal the sum of column

33 multiplied by $\frac{\sum y \Delta}{\sum \Delta}$ and the sum of column 40

should equal $-100 dx$. M_o in column 41 is computed by taking the algebraic sum of the corresponding terms in columns 38, 39, and 40. The algebraic sum of column 41 should equal the algebraic sum of columns 38, 39, and 40.

TABLE 4.—Computations for M_o .

31	32	34	35	36	37	38	39	40	41	
Points	z	H_o	V_o	Δ	$\frac{1}{2} \Sigma \Delta$	$\frac{1}{2} \Sigma (z-k) \Delta$	$\frac{dx}{\Sigma \Delta}$ Col. 37	$\frac{\Sigma y \Delta}{\Sigma \Delta}$	$-20 \frac{dx}{2} V_o$	M_o
0	0									
1	1	0.000	1.0000	4.7	472.9	4,537.2	28.500	0.000	30.000	-1.500
2	3	.062	.99556	10.1	462.8	4,064.3	25.529	.363	29.867	-3.975
3	5	.219	.98256	16.2	446.6	3,601.5	22.622	1.282	29.477	-5.573
4	7	.478	.95748	21.3	425.3	3,154.9	19.817	2.799	28.724	-6.108
5	9	.815	.91861	26.0	399.3	2,729.6	17.146	4.772	27.558	-5.640
6	11	1.191	.86551	29.6	369.7	2,330.3	14.637	6.974	25.965	-4.354
7	13	1.561	.79915	31.8	337.9	1,960.6	12.315	9.141	23.974	-2.518
8	15	1.878	.72171	31.7	306.2	1,622.7	10.193	10.997	21.651	-0.461
9	17	2.111	.63638	33.7	272.5	1,316.5	8.269	12.362	19.091	+1.540
10	19	2.232	.54602	33.7	238.8	1,044.0	6.558	13.070	16.381	+3.247
10'	21	2.232	.45398	33.7	205.1	805.2	5.058	13.070	13.619	+4.569
9'	23	2.111	.36362	33.7	171.4	600.1	3.769	12.362	10.909	+5.222
8'	25	1.878	.27829	31.7	139.7	428.7	2.693	10.997	8.349	+5.341
7'	27	1.561	.20085	31.8	107.9	289.0	1.815	9.141	6.026	+4.930
6'	29	1.191	.13449	29.6	78.3	181.1	1.137	6.974	4.035	+4.076
5'	31	.815	.08139	26.0	52.3	102.8	.646	4.772	2.442	+2.976
4'	33	.478	.04252	21.3	31.0	50.5	.317	2.799	1.276	+1.840
3'	35	.219	.01744	16.2	14.8	19.5	.122	1.282	.523	+.881
2'	37	.062	.00444	10.1	4.7	4.7	.030	.363	.133	+.260
1'	39	0.000	0.0000	4.7	0.0	0.0	0.000	0.000	0.000	0.000
0'	40									
		21.094	10.0000				181.173	123.520	-300.000	+4.693

$$M_o = \frac{dx}{\Sigma \Delta} \frac{1}{2} \Sigma (z-k) \Delta + H_o \frac{\Sigma y \Delta}{\Sigma \Delta} - 20 \frac{dx}{2} V_o$$

$$\frac{dx}{\Sigma \Delta} = .0062814 \quad -20 \frac{dx}{2} = -30.0$$

For check:

$$\frac{\Sigma y \Delta}{\Sigma \Delta} \Sigma \text{Col. 33} = 123.523 = \Sigma \text{Col. 39.}$$

$$-20 \frac{dx}{2} \Sigma \text{Col. 34} = -300.0 = \Sigma \text{Col. 40.}$$

$$\Sigma \text{Col. 38} + \Sigma \text{Col. 39} + \Sigma \text{Col. 40} = +4.693 = \Sigma \text{Col. 41.}$$

Tables 5, 6, and 7.—Computation of bending moments at points 3, 8, and $10\frac{1}{2}$.—We now have the values of H_o , V_o , and M_o for a load of unity placed at any point on the arch. It must now be decided at which other points the stresses are desired. Usually three other points are sufficient, and they should be selected with care in order to get the critical points. One should be at or near the crown, one near the haunch, and probably the best place for the third is between the haunch and the left abutment. Points 3 and 8 are chosen because they are near changes in the reinforcement; and point $10\frac{1}{2}$, being the crown, is also chosen.

Columns 46 to 50, inclusive, are used in finding the moment at point 3, from the formula $M_3 = M_o - H_o y + m_3$. In computing the moment at any particular point,

z and y are constant, while k varies with the position of the unit load. We now desire the moment at a particular point caused by a load at each point. For point 3 the value of z is 5 and the value of y can be found in column 20. These figures should be set down in the spaces indicated in the heading for columns 46 to 50. For convenience H_o , V_o , and M_o are copied in columns 43, 44, and 45. The values of m_x are computed from the formula $m_x = \left[V_o z - (z-k) \right] \frac{dx}{2}$ where k is less than z , and from the formula $m_x = V_o z \frac{dx}{2}$ where k is greater than z . Column 46 is computed by multiplying the values of V_o by z ($z=5$ for point 3). It is only necessary to fill out column 46 for a load at those points where

TABLE 5.—Computations for M_3 TABLE 6.—Computations for M_8 TABLE 7.—Computations for $M_{10\frac{1}{2}}$

42	43	44	45	46	47	48	49	50	51	52	53	54	55	56	57	58	59	60
Points	H_o	V_o	M_o	Point 3, $z_3=5$; $y_3=3.51$					Point 8, $z_8=15$; $y_8=6.85$					Point $10\frac{1}{2}$, $z_{10\frac{1}{2}}=20$; $y_{10\frac{1}{2}}=7.21$				
				$V_{o z_3}$	$V_{o z_3}-(z_3-k)$	m_3	$-H_o y_3$	M_3	$V_{o z_8}$	$V_{o z_8}-(z_8-k)$	m_8	$-H_o y_8$	M_8	$V_{o z_{10\frac{1}{2}}}$	$V_{o z_{10\frac{1}{2}}}-(z_{10\frac{1}{2}}-k)$	$m_{10\frac{1}{2}}$	$-H_o y_{10\frac{1}{2}}$	$M_{10\frac{1}{2}}$
0																		
1	0.000	1.00000	-1.500	5.000	1.0000	1.500	0.000	0.000	15.000	1.0000	1.500	0.000	0.000	20.000	1.000	1.500	0.000	0.000
2	.062	.99556	-3.975	4.978	2.978	4.467	-.218	+.274	14.933	2.933	4.400	-.425	+.000	19.911	2.911	4.367	-.447	+.055
3	.219	.98256	-5.573			7.369	-.769	+.1027	14.738	4.738	7.107	-1.500	+.034	19.651	4.651	6.976	-1.579	+.176
4	.478	.95748	-6.108			7.181	-1.678	+.605	14.362	6.362	9.543	-3.274	+.161	19.150	6.150	9.225	-3.446	+.329
5	.815	.91861	-5.640			6.890	-2.861	-1.611	13.779	7.779	11.669	-5.583	+.446	18.372	7.372	11.058	-5.876	+.458
6	1.191	.86551	-4.354			6.491	-4.180	-2.043	12.983	8.983	13.475	-8.158	+.963	17.310	8.310	12.465	-8.587	+.476
7	1.561	.79915	-2.518			5.994	-5.479	-2.003	11.987	9.987	14.981	-10.693	+.1.770	15.983	8.983	13.475	-11.255	+.298
8	1.878	.72171	-0.461			5.413	-6.592	-1.640			16.238	-12.864	+.2.913	14.434	9.434	14.151	-13.540	+.150
9	2.111	.63638	+1.540			4.773	-7.409	-1.096			14.319	-14.460	+.1.399	12.728	9.728	14.592	-15.220	+.912
10	2.232	.54602	+3.247			4.095	-7.834	-.492			12.285	-15.289	+.243	10.920	9.920	14.880	-16.093	+2.034
10'						3.405	-7.834	+.080			10.215	-15.289	-.565			13.619	-16.093	+2.035
9'						2.727	-7.409	+.540			8.181	-14.460	-1.057			10.909	-15.220	+.911
8'						2.087	-6.592	+.836			6.262	-12.864	-1.261			8.349	-13.540	+.150
7'						1.506	-5.479	+.957			4.519	-10.693	-1.244			6.025	-11.255	+.300
6'						1.009	-4.180	+.905			3.026	-8.158	-1.056			4.035	-8.587	+.476
5'						.610	-2.861	+.725			1.831	-5.583	-.776			2.442	-5.876	+.458
4'						.319	-1.678	+.481			.957	-3.274	-.477			1.276	-3.446	+.330
3'						.131	-.769	+.243			.392	-1.500	-.227			.523	-1.579	+.175
2'						.033	-.218	+.075			.100	-.425	-.065			.133	-.447	+.054
1'	0.000	0.00000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
0'	21.094	10.00000	+4.693			66.000	-74.040	-3.347			141.000	-144.492	+1.201			150.000	-152.086	+2.607

$$M_x = M_o + m_x - H_o y$$

$$m_x = \left[V_o z_x - (z_x - k) \right] \frac{dx}{2}$$

For check:

$$\Sigma \text{Col. 45} + \Sigma \text{Col. 48} + \Sigma \text{Col. 49} = \Sigma \text{Col. 50.}$$

$$\Sigma \text{Col. 45} + \Sigma \text{Col. 53} + \Sigma \text{Col. 54} = \Sigma \text{Col. 55.}$$

$$\Sigma \text{Col. 45} + \Sigma \text{Col. 58} + \Sigma \text{Col. 59} = \Sigma \text{Col. 60.}$$

k is less than z . Column 47 is computed by subtracting $(z-k)$ from the figures in column 46. The values of m_x may now be computed for a load at those points where k is less than z by multiplying the figures in column 47 by $\frac{dx}{2}$ and for the remaining points by multiplying the values of V_o by $z\frac{dx}{2}$.

The value of $-H_o y$ in column 49 is computed by multiplying each value of H_o by $-y$ as given in the heading. The bending moment $M_3 = M_o + m_3 - H_o y$ is now computed by adding algebraically the corresponding values of columns 45, 48, and 49 and the results tabulated in column 50. The bending moments at the two other points are computed in the same way, using columns 51 to 60, inclusive. We now have in the tabulation which includes Tables 5, 6, and 7 (sheet 4 of assembly of data) the vertical shear, the horizontal thrust, and the bending moment at point 0 and the bending moment at three other points caused by a load of unity at each of the points of the arch ring.

Table 8.—Computations for the dead-load moments, thrust, and shears.—The values of H_o , V_o , M_o and the unit-load bending moments at the points 3, 8, and $10\frac{1}{2}$ are copied in columns 62 to 67 of Table 8. The dead load which is applied at each point is then computed and tabulated in column 68. This may be conveniently tabulated in a supplementary table as follows: The dead load at any point is equal to $150hds + 110dx h_f$, in which the weight of the concrete is assumed to be 150 pounds per cubic foot and the weight of the earth fill 110 pounds per cubic foot. Values of h and ds are found in columns 2 and 10, respectively, of Table 1, and h_f is the depth of the fill above the arch ring and dx for this arch is 3 feet.

TABLE 8.—Computations of H , V , and M for dead load

61	62	63	64	65	66	67	68	69	70	71	72	73	74
Point	Unit loads						D. L.	Dead load					
	H_o	V_o	M_o	M_3	M_8	$M_{10\frac{1}{2}}$		H_o	V_o	M_o	M_3	M_8	$M_{10\frac{1}{2}}$
1	0	1.00000	- 1.500	0	0	0	3,000	0	3,000	- 4,500	0	0	0
2	.062	.99556	- 3.975	+ .274	+ .000	- .055	2,330	144	2,320	- 9,262	+ 638	0	- 128
3	.219	.98256	- 5.573	+ 1.027	+ .034	- .176	1,880	412	1,847	- 10,477	+ 1,931	+ 64	- 331
4	.478	.95748	- 6.108	+ .605	+ .161	- .329	1,540	736	1,475	- 9,406	+ 932	+ 248	- 507
5	.815	.91861	- 5.640	+ 1.611	+ .446	- .458	1,250	1,019	1,148	- 7,050	+ 2,014	+ 558	- 573
6	1.191	.86551	- 4.354	+ 2.043	+ .963	- .476	1,040	1,239	900	- 4,528	+ 2,125	+ 1,002	- 495
7	1.561	.79915	- 2.518	+ 2.003	+ 1.770	- .298	860	1,342	687	- 2,165	+ 1,723	+ 1,522	- 256
8	1.878	.72171	- 0.461	+ 1.640	+ 2.913	+ .150	750	1,409	541	- 346	+ 1,230	+ 2,185	+ 113
9	2.111	.63638	+ 1.540	+ 1.096	+ 1.399	+ .912	680	1,435	433	+ 1,047	+ 745	+ 951	+ 620
10	2.232	.54602	+ 3.247	+ .492	+ .243	+ 2.034	670	1,495	366	+ 2,175	+ 330	+ 163	+ 1,363
10'	2.232	.45398	+ 4.509	+ .080	- .565	+ 2.035	670	1,495	304	+ 3,021	+ 54	- 379	+ 1,363
9'	2.111	.36362	+ 5.222	+ .540	- 1.057	+ .911	680	1,435	247	+ 3,551	+ 367	- 719	+ 619
8'	1.878	.27829	+ 5.341	+ .836	- 1.261	+ .150	750	1,409	209	+ 4,006	+ 627	- 946	+ 113
7'	1.561	.20085	+ 4.930	+ .957	- 1.244	+ .300	860	1,342	173	+ 4,240	+ 823	- 1,070	- 258
6'	1.191	.13149	+ 4.076	+ .905	- 1.056	- .476	1,040	1,239	140	+ 4,239	+ 941	- 1,098	- 495
5'	.815	.08139	+ 2.976	+ .725	- .776	- .458	1,250	1,019	102	+ 3,720	+ 906	- 970	- 573
4'	.478	.04252	+ 1.840	+ .481	- .477	- .330	1,540	736	65	+ 2,834	+ 741	- 735	- 508
3'	.219	.01744	+ .881	+ .243	- .227	- .175	1,880	412	33	+ 1,656	+ 457	- 427	- 329
2'	.062	.00444	+ .260	+ .075	- .065	- .054	2,330	144	10	+ 606	+ 175	- 151	- 126
1'	0	0	0	0	0	0	3,000	0	0	0	0	0	0
	21.094	10.0000	+ 34.822	+ 6.143	+ 7.929	+ 6.192	28,000	18,462	14,000	+ 31,095	+ 7,660	+ 6,693	+ 4,191
			- 30.129	- 9.490	- 6.728	- 3.585				- 47,734	- 9,099	- 6,495	- 4,579
			+ 4.693	- 3.347	+ 1.201	+ 2.607				- 16,639	- 1,439	+ 198	- 388

$$M_t = -H_t \left(y - \frac{\Sigma y \Delta}{\Sigma \Delta} \right)$$

At point 0, $M_t = - \left\{ \begin{smallmatrix} +2,762 \\ -3,682 \end{smallmatrix} \right\} \times (-5.856) = \left\{ \begin{smallmatrix} +16,174 \\ -21,562 \end{smallmatrix} \right\}$

At point 3, $M_t = - \left\{ \begin{smallmatrix} +2,762 \\ -3,682 \end{smallmatrix} \right\} \times (-2.346) = \left\{ \begin{smallmatrix} +6,480 \\ -8,638 \end{smallmatrix} \right\}$

At point 8, $M_t = - \left\{ \begin{smallmatrix} +2,762 \\ -3,682 \end{smallmatrix} \right\} \times (+0.994) = \left\{ \begin{smallmatrix} +2,745 \\ +3,660 \end{smallmatrix} \right\}$

At point $10\frac{1}{2}$, $M_t = - \left\{ \begin{smallmatrix} +2,762 \\ -3,682 \end{smallmatrix} \right\} \times (+1.364) = \left\{ \begin{smallmatrix} -3,470 \\ +4,985 \end{smallmatrix} \right\}$

Computation of dead load

Point	hds	$150hds$	h_f	$330h_f$	Dead load
					Pounds
1.....	6.79	1,020	6.00	1,980	3,000
2.....	4.96	744	4.80	1,585	2,329
3.....	4.11	616	3.82	1,260	1,876
4.....	3.69	553	3.00	990	1,543
5.....	3.40	510	2.25	742	1,252
6.....	3.20	480	1.70	560	1,040
7.....	3.08	462	1.20	396	858
8.....	3.04	456	.90	297	753
9.....	3.00	450	.70	232	682
10.....	3.00	450	.67	220	670

Column 69 to 74, inclusive, are computed by multiplying the dead load in column 68 by the proper figure from columns 62 to 67, inclusive. The algebraic sums of columns 69, 70, and 71 are the horizontal thrust, vertical shear, and bending moment, respectively, at point 0 resulting from the dead load. The algebraic sums of columns 72, 73, and 74 are the dead-load bending moments at points 3, 8, and $10\frac{1}{2}$.

Table 9.—Computation of stresses.—Column 75 in Table 9 is now to be filled out. The values of $\cos \phi$ for each point desired are taken from column 28. The values of $\sin \phi$ may be found by scaling the drawing. Since the stresses are desired in pounds per square inch, and all previous dimensions have been in feet, the area in column 75 should be 144 times h (taken from column 2) plus n times the area of steel in square inches. The values of $\frac{h}{2I}$ are computed from columns 2 and 9 and this result should be divided by 144 to get results in pounds per square inch. The values of M , H , and V for the dead load are taken from the sums of columns 69 to 74, inclusive, and set down in the proper places in

TABLE 9.—*Computation of maximum stresses*

		76	77	78	79	80	81	82	83	84	85	86	87	88	89	90													
												Extrados	Intrados	Maximum stresses															
			<i>H</i>	<i>M</i>	<i>V</i>	<i>H</i> cos ϕ	<i>V</i> sin ϕ	<i>N</i>	$\frac{N}{A}$	$M \frac{h}{2I}$	$\frac{N}{A} + M \frac{h}{2I}$	$\frac{N}{A} - M \frac{h}{2I}$		Extrados		Intrados													
														+	-	+	-												
Point 0	D. L.	+	18,462	-	16,639	14,000	+	15,471	7,672	+	23,142	+	66	-	118	-	52	+	184	-	52	-	52	+	184	+	184		
sin $\phi = .548$	C. L. L.	+		+			+			+		+																	
cos $\phi = .838$	U. L. L.	+	5,584	+	13,058	1,035	+	4,679	567	+	5,246	+	15	+	93	+	108	-	78	+	108	-					-	78	
Area = 348.	C. L. L.	+		+			+			+		+																	
$\frac{h}{2I} = .00709$	U. L. L.	+	2,326	-	11,298	2,715	+	1,949	1,488	+	3,437	+	10	-	80	-	70	+	90	+	122	-					-	108	
	T.	+	2,762	+	16,174	0	+	2,315	0	+	2,315	+	7	+	115	+	122	+	108	+	122	+					-		
	T.	-	3,682	-	21,562	0	-	3,086	0	-	3,086	-	9	-	153	-	162	+	144	-							-		
Total excluding temperature																	Total												
																	+	178	+	71	+	71	+					-	2
Point 3	D. L.	+	18,462	-	1,439	8,670	+	17,465	2,982	+	20,447	+	104	-	33	+	71	+	137	+	137	+					-	137	
sin $\phi = .344$	C. L. L.	+		+			+			+		+																	
cos $\phi = .946$	U. L. L.	+	4,060	+	2,304	958	+	3,841	330	+	4,171	+	21	+	53	+	74	-	32	+	74	-					-	32	
Area = 196.	C. L. L.	+		+			+			+		+																	
$\frac{h}{2I} = .02303$	U. L. L.	+	3,850	-	3,559	2,042	+	3,642	762	+	4,344	+	22	-	82	-	60	+	104	+	162	-					-		
	T.	+	2,762	+	6,480	0	+	2,613	0	+	2,613	+	13	+	149	+	162	+	136	+	162	+					-	136	
	T.	-	3,682	-	8,638	0	-	3,483	0	-	3,483	-	18	-	199	-	217	+	181	-							-		
Total excluding temperature																	Total												
																	+	145	+	307	+	307	+					-	31
Point 8	D. L.	+	18,462	+	198	2,100	+	18,388	216	+	18,604	+	120	+	7	+	127	+	113	+	127	+					+	113	
sin $\phi = .103$	C. L. L.	+		+			+			+		+																	
cos $\phi = .996$	U. L. L.	+	3,955	+	2,973	534	+	3,939	55	+	3,994	+	26	+	111	+	137	-	85	+	137	-					-	85	
Area = 155.	C. L. L.	+		+			+			+		+																	
$\frac{h}{2I} = .03728$	U. L. L.	+	3,955	-	2,523	591	+	3,939	61	+	4,000	+	26	-	94	-	68	+	120	+	162	-					-		
	T.	+	2,762	-	2,745	0	+	2,751	0	+	2,751	+	18	-	162	-	84	+	120	+	162	-					-		
	T.	-	3,682	+	3,660	0	-	3,667	0	-	3,667	-	24	+	136	+	112	+	160	+	112	+					-	160	
Total excluding temperature																	Total												
																	+	264	+	376	+	376	+					-	132
Point 10½	D. L.	+	18,462	-	388	0	+	18,462	0	+	18,462	+	121	-	15	+	106	+	136	+	106	+					+	136	
sin $\phi = 0$	C. L. L.	+		+			+			+		+																	
cos $\phi = 1.00$	U. L. L.	+	4,666	+	2,322	0	+	4,666	0	+	4,666	+	31	+	91	+	122	-	60	+	122	-					-	60	
Area = 152.	C. L. L.	+		+			+			+		+																	
$\frac{h}{2I} = .03901$	U. L. L.	+	3,245	-	1,314	0	+	3,245	0	+	3,245	+	21	-	52	-	31	+	73	+	153	-					-		
	T.	+	2,762	-	3,470	0	+	2,762	0	+	2,762	+	18	-	135	-	117	+	153	+	170	-					-		
	T.	-	3,682	+	4,985	0	-	3,682	0	-	3,682	-	24	+	194	+	170	+	218	+	170	+					-	218	
Total excluding temperature																	Total												
																	+	228	+	398	+	398	+					-	142

$$N = H \cos \phi + V \sin \phi$$

$$f_c = \frac{N}{A} \pm M \frac{h}{2I}$$

columns 77, 78, and 79. It should be remembered that the V_0 given in the sum of column 70 is the dead-load vertical shear at point 0, and to get the vertical shear at any other point the loads between point 0 and the point under consideration must be subtracted from V_0 .

We will consider next the live-load thrusts, moments, and shears. A uniform live load of 125 pounds per square foot has been assumed, and since we are considering a strip of the arch ring 1 foot wide, the live load at one point is $125dx$, or 375 pounds. We wish to find the maximum stress at each of the points, 0, 3, 8, and $10\frac{1}{2}$ and the stress is a function of the moment, thrust, and shear. It is not necessarily maximum at the same time that the moment is, but it is so nearly so that this is assumed to be true. Inspecting column 64 it is seen that the maximum positive moment at point 0 is produced by placing a live load at all of the points which give a positive moment, and no load should be placed at the points which give a negative moment. Thus the maximum positive live-load moment at point 0 is found by multiplying the sum of all of the positive quantities in column 64 by 375 pounds as is done in the table following.

The horizontal thrust which occurs at the same time as this maximum moment is found by adding the quantities in column 62 for the points which give a positive moment and multiplying the sum by 375. The same procedure is followed for the vertical shear, using the quantities in column 63. The maximum negative moment and the thrust and shear which occur at the same time are found by placing the loads at all of the points which give negative moments. The live-load

moments, thrusts, and shears at the other points are found in the same way, but it should be remembered that the values of V_0 in column 63 are the values of the reaction at the left support, and if there are any loads to the left of the point under consideration the shear is found by adding the proper quantities in the V_0 -column and then subtracting 1.00 for each load to the left. Then the quantity thus obtained is multiplied by the live load per load point as shown in the following computations:

MOMENTS, THRUSTS, AND SHEARS DUE TO LIVE LOAD

Live load = 125 pounds per square foot
 = $125 \times 3 = 375$ pounds per load point

Point 0:

$$\begin{aligned} +M &= 375 \times 34.822 = +13,058 \\ H &= 375 \times 14.890 = 5,584 \\ V &= 375 \times 2.759 = 1,035 \\ -M &= 375 \times 30.129 = -11,298 \\ H &= 375 \times 6.204 = 2,326 \\ V &= 375 \times 7.241 = 2,715 \end{aligned}$$

Point 3:

$$\begin{aligned} +M &= 375 \times 6.143 = +2,304 \\ H &= 375 \times 10.828 = 4,060 \\ V &= 375 \times (4.555 - 2) = 958 \\ -M &= 375 \times 9.490 = -3,559 \\ H &= 375 \times 10.266 = 3,850 \\ V &= 375 \times 5.445 = 2,042 \end{aligned}$$

Point 8:

$$\begin{aligned} +M &= 375 \times 7.929 = +2,973 \\ H &= 375 \times 10.547 = 3,955 \\ V &= 375 \times (8.423 - 7) = 534 \\ -M &= 375 \times 6.728 = -2,523 \\ H &= 375 \times 10.547 = 3,955 \\ V &= 375 \times 1.577 = 591 \end{aligned}$$

Point 10½:

$$\begin{array}{rcl}
 +M & = 375 \times 6.192 & = +2,322 \\
 H & = 375 \times 12.442 & = 4,666 \\
 V & = 375 \times (3-3) & = 0 \\
 -M & = 375 \times 3.585 & = -1,344 \\
 H & = 375 \times 8.652 & = 3,245 \\
 V & = 375 \times (7-7) & = 0
 \end{array}$$

The horizontal thrust due to a change in temperature is computed from the formula $H_t = \frac{20 \epsilon t E_c}{C}$. If E_c be taken as 2,000,000 pounds per square inch or 144 x 2,000,000 pounds per square foot and ϵ be taken as 0.000006, the formula becomes $H_t = \frac{34560}{C} t$. H_t is constant for all points of the arch ring, and its value is calculated and recorded in column 77. M_t is computed from the formula $M_t = -H_t \left(y - \frac{\sum y \Delta}{\sum \Delta} \right)$. Values of $\left(y - \frac{\sum y \Delta}{\sum \Delta} \right)$ are found in column 23 except for point 0 and the value for this point can be quickly computed. The value of V_t is zero in every case.

$H \cos \phi$ and $V \sin \phi$ are computed and tabulated in columns 80 and 81, respectively. N in column 82 is computed by adding the corresponding values in columns 80 and 81. Column 83 is computed by dividing N in column 82 by A in column 75. M_{2I}^h in column 84 is computed by multiplying M in column 78 by $\frac{h}{2I}$. The stress in the concrete at the extrados $\left(\frac{N}{A} + M_{2I}^h \right)$ is computed for column 85 by adding the corresponding values in columns 83 and 84. The stress in the concrete at the intrados is computed in column 86 by subtracting column 84 from column 83, observing the correct algebraic sign.

Columns 85 and 86 give the stresses due to various conditions of loading and temperature, but those conditions do not all obtain at one time. To find the maximum stress which may obtain, columns 87 to 90 are used, as indicated in the headings. For each case the maximum stress is caused by a combination of the dead load, one position of the live load, and one condition of the temperature. These stresses are tabulated in columns 87 to 90, inclusive, and the algebraic sum taken. The correct algebraic sign must be observed in all cases.

These maximum stresses have been computed upon the assumption that the arch ring is uniformly elastic and acts in accordance with Hook's law under all conditions—that is, we have assumed that the concrete will withstand the tensile stresses as well as the compressive. If the computed tensile stresses do not exceed the allowable tensile stress for concrete, the above assumption is practically true, but if we find that the tension in the concrete exceeds the allowable stress at any point the assumption is not true and we must assume that the concrete cracks at that point and all of the tension is taken by the steel reinforcement. The stresses in the steel and concrete at that point should then be computed by another method.

This may be done by dividing the maximum moment by the maximum thrust (columns 78 and 82) to find the eccentricity of the thrust and then computing the stresses by the theory of bending and direct stress in reinforced concrete for which formulas and diagrams may be found in any textbook on reinforced concrete.

Stresses in steel and concrete.—Columns 87 to 90, inclusive, give the stresses in the concrete computed

upon the assumption that the concrete will withstand the stress to which it is subjected. It is found that the tension in the concrete at the abutment under certain conditions is 284 pounds per square inch. Under these circumstances the concrete will crack and all of the tension must be taken by the steel.



A TWO-RIBBED OPEN-SPANDREL ARCH DESIGNED BY THE METHOD DESCRIBED HERE BUT WITH SOME VARIATION ON ACCOUNT OF THE POSITION OF LOAD POINTS AS FIXED BY THE LOCATION OF COLUMNS

When the tensile strength of the concrete is exceeded, the stresses in the steel and concrete may be found from the diagram on page 405 of Hool and Johnson's "Concrete Engineers' Handbook." It will be noted that in this case there is reinforcement on the tension side only. The nomenclature used in the following computations, if not previously explained, is found on page 403 of Hool and Johnson.

At point 0:

	M	N
D. L. =	-16,639	+23,142
L. L. =	-11,298	+3,437
T. =	-21,562	-3,086
	-49,499	+23,493
$u = \frac{-49,499}{+23,493}$	$= -2.11$ ft.	

$$e' = u + \left(\frac{h}{2} - d' \right) = 2.11 + 0.98 = 3.09 \text{ ft.} = 37.1 \text{ in.}$$

$$d = h - d' = 2.30 - 0.17 = 2.13 \text{ ft.} = 25.6 \text{ in.}$$

$$\frac{e'}{d} = \frac{37.1}{25.6} = 1.45.$$

Area of steel = 1.125 sq. in. (see fig. 5):

$$p = \frac{1.125}{144 \times 2.13} = 0.0037 \text{ per cent steel.}$$

$$K = \frac{N e'}{b d^2} = \frac{23,493 \times 37.1}{12 \times (25.6)^2} = 111.$$

From diagram (Hool and Johnson) $f_c = 630$ and $f_s = 14,000$.

Therefore the arch is satisfactory at this point.

At point 3 the worst conditions of loading and temperature give 206 pounds tension in the concrete, which will cause the concrete to crack.

At point 3:

	M	N
D. L. =	-1,439	+20,446
L. L. =	-3,559	+4,344
T. =	-8,638	-3,483
	-13,636	+21,308
$u = \frac{-13,636}{+21,308}$	$= 0.64$ ft.	

$$e' = 0.64 + 0.48 = 1.12 \text{ ft.} = 13.4 \text{ in.}$$

$$d = 13.6 \text{ in.}$$

$$\frac{e'}{d} = \frac{13.4}{13.6} = 0.985.$$

$$p = \frac{0.5625}{144 \times 1.13} = 0.00346 \text{ per cent steel.}$$

$$K = \frac{N e'}{b d^2} = \frac{21,305 \times 13.4}{12 \times (13.6)^2} = 128.$$

From diagram, $f_c = 575$ and $f_s = 7,500$.

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- Vol. 5, No. 17, D- 2. Effect of Controllable Variables upon the Penetration Test for Asphalts and Asphalt Cements.
- Vol. 5, No. 19, D- 3. Relation Between Properties of Hardness and Toughness of Road-Building Rock.
- Vol. 5, No. 24, D- 6. A New Penetration Needle for Use in Testing Bituminous Materials.
- Vol. 6, No. 6, D- 8. Tests of Three Large-Sized Reinforced-Concrete Slabs Under Concentrated Loading.
- Vol. 10, No. 5, D-12. Influence of Grading on the Value of Fine Aggregate Used in Portland Cement Concrete Road Construction.
- Vol. 11, No. 10, D-15. Tests of a Large-Sized Reinforced-Concrete Slab Subjected to Eccentric Concentrated Loads.

* Department supply exhausted.

UNITED STATES DEPARTMENT OF AGRICULTURE

BUREAU OF PUBLIC ROADS

STATUS OF FEDERAL AID HIGHWAY CONSTRUCTION

AS OF

MAY 31, 1927

STATES	FISCAL YEARS 1917-1926				FISCAL YEAR 1927				BALANCE OF FEDERAL AID FUND AVAILABLE FOR NEW PROJECTS			
	PROJECTS COMPLETED PRIOR TO JULY 1, 1926				* PROJECTS UNDER CONSTRUCTION				PROJECTS APPROVED FOR CONSTRUCTION			
	TOTAL COST	FEDERAL AID	MILES		TOTAL COST	FEDERAL AID	MILES		ESTIMATED COST	FEDERAL AID ALLOTTED	MILES	
Alabama	\$ 18,226,411.34	\$ 8,725,985.09	1298.3		\$ 1,624,950.34	\$ 889,114.85	101.9		\$ 273,351.16	\$ 136,675.87	14.3	
Arizona	10,949,476.25	5,863,772.35	729.8		766,036.23	585,632.71	48.9		735,979.00	608,902.82	31.2	
Arkansas	18,364,544.50	7,662,698.35	1323.0		3,972,470.13	1,858,494.40	237.6		925,746.03	262,746.03	18.0	
California	27,142,596.90	13,043,592.30	1058.0		7,797,030.13	3,851,460.35	243.2		182,512.27	99,131.19	6.5	
Colorado	13,905,904.64	7,127,288.18	745.0		1,533,319.76	774,541.81	81.1		35,740.70	20,057.14	2.5	
Connecticut	5,414,567.19	2,100,595.80	117.1		982,825.10	343,414.74	20.1		1,245,486.94	230,040.13	13.0	
Delaware	4,919,052.29	1,761,665.60	124.3		1,314,974.26	583,906.92	35.2		271,632.15	63,316.77	8.4	
Florida	3,632,680.26	1,824,322.32	132.9		3,654,176.05	1,803,950.13	112.2		1,150,401.52	358,051.76	24.2	
Georgia	24,751,506.97	11,664,237.86	1754.0		5,915,181.39	2,869,016.13	338.3		855,217.07	400,661.56	42.1	
Idaho	11,051,193.14	5,982,113.70	724.7		2,164,317.31	1,193,414.46	110.8		5,400,226.91	2,656,826.04	194.3	
Illinois	44,116,511.86	20,915,995.74	1377.7		4,432,370.30	2,165,526.95	155.9		2,027,795.86	1,077,570.98	73.2	
Indiana	16,949,425.87	8,172,132.19	534.3		5,659,150.69	2,694,046.53	177.7		3,170,180.51	1,270,144.71	96.1	
Iowa	29,053,975.40	11,920,362.10	2114.8		5,015,643.90	2,436,134.60	353.4		2,603,194.84	1,016,180.99	194.1	
Kansas	34,925,301.60	12,182,573.25	1163.5		1,395,681.55	622,222.12	102.8		756,162.00	387,426.88	47.3	
Kentucky	20,737,706.10	8,932,082.25	758.3		2,203,656.73	315,591.55	100.1		26,200.56	13,100.33	3.0	
Louisiana	13,630,522.68	6,144,739.99	1085.9		2,046,952.92	949,152.32	122.8		744,132.14	215,406.00	17.0	
Maine	8,142,543.76	1,182,507.35	303.3		1,817,427.30	665,945.58	54.0		1,238,679.25	589,300.00	59.0	
Maryland	10,324,945.10	5,121,591.22	423.3		855,260.62	411,377.47	54.5		774,046.27	190,341.12	10.9	
Massachusetts	18,346,757.71	6,657,260.62	374.5		2,316,488.31	768,667.53	35.9		1,631,486.50	755,705.00	40.5	
Michigan	25,937,240.78	11,827,052.30	933.0		5,443,350.45	2,311,166.69	121.0		1,204,531.98	525,900.00	48.5	
Minnesota	37,170,985.95	15,966,116.56	3181.9		7,958,662.52	3,460,029.11	461.6		1,088,722.76	530,445.75	72.0	
Mississippi	15,146,088.52	7,414,534.10	1129.0		2,522,270.42	1,258,294.62	165.2		1,286,312.79	599,394.16	52.5	
Missouri	26,969,166.92	13,736,014.85	1543.2		12,095,884.16	5,322,158.35	359.8		2,973,536.94	1,485,232.10	131.1	
Montana	11,400,983.81	6,333,465.89	1054.9		1,454,011.91	953,822.80	95.7		335,952.79	157,000.00	17.0	
Nebraska	11,533,401.62	5,474,202.52	1768.3		4,356,776.52	2,142,485.17	440.9		1,485,232.10	744,132.14	59.0	
Nevada	7,558,195.51	5,130,934.59	538.8		2,776,355.76	2,382,094.86	301.0		335,675.36	147,698.10	8.9	
New Hampshire	4,992,598.60	2,377,450.07	237.6		876,339.16	401,477.98	27.2		1,699,048.74	433,065.00	28.9	
New Jersey	16,346,301.01	5,098,342.21	290.3		5,881,939.07	2,397,022.27	26.0		9,792,833.00	2,227,927.50	130.0	
New Mexico	12,404,337.77	7,339,657.38	1427.0		912,427.18	587,767.43	78.2		1,006,095.75	432,169.75	26.5	
New York	43,224,279.79	17,911,957.19	1197.0		10,788,701.76	3,701,463.46	236.9		1,871,939.41	930,501.55	271.4	
North Carolina	27,009,415.47	11,177,337.94	1257.9		8,266,428.74	3,341,565.22	202.2		714,896.16	244,093.22	13.0	
North Dakota	12,013,311.40	6,031,699.78	2193.1		3,559,649.54	1,710,135.10	522.4		1,713,513.16	766,641.99	126.3	
Ohio	47,699,522.90	17,371,787.03	1364.1		4,921,858.93	1,959,598.73	150.8		360,519.22	196,950.85	25.0	
Oklahoma	28,247,950.33	13,159,999.15	1178.9		2,134,006.75	957,590.06	85.8		2,809,736.18	927,063.64	57.3	
Oregon	17,027,782.42	8,593,214.79	939.2		2,288,382.35	1,314,004.60	105.6		1,713,513.16	766,641.99	126.3	
Pennsylvania	61,366,150.80	21,550,732.04	1188.8		14,599,594.29	4,240,910.49	311.3		2,909,736.18	927,063.64	57.3	
Rhode Island	3,988,616.09	1,589,899.06	86.7		1,244,797.39	439,650.00	29.3		3,503,402.56	235.9		
South Carolina	15,020,539.90	6,766,322.93	1481.9		1,581,400.03	760,665.87	86.5		1,277,631.75	231,100.29	15.8	
South Dakota	17,468,373.19	8,603,826.97	2181.2		1,753,690.05	903,698.57	321.7		645,120.64	357,016.32	128.5	
Tennessee	21,624,531.57	10,276,584.02	780.0		2,552,757.67	1,249,742.90	78.5		1,277,631.75	231,100.29	15.8	
Texas	69,831,721.18	27,424,747.75	4929.4		8,802,957.56	4,036,746.78	537.1		6,246,764.95	2,931,912.31	53.4	
Utah	8,253,178.03	5,099,440.68	546.4		901,199.20	668,639.27	62.5		566,014.46	253,912.69	30.1	
Vermont	4,242,042.64	2,017,699.51	134.5		795,075.59	331,156.50	18.2		873,280.34	329,018.91	21.5	
Virginia	21,990,249.44	10,395,728.11	1005.5		2,064,707.13	2,064,707.13	157.6		1,071,311.26	65,000.00	21.9	
Washington	17,078,511.63	7,782,953.46	668.5		4,634,607.19	4,634,607.19	42.5		231,489.59	1,750,500.00	10.2	
West Virginia	9,473,716.44	4,141,062.65	392.9		951,130.68	432,685.36	26.5		109,976.65	54,988.32	5.7	
Wisconsin	24,856,508.19	10,392,705.73	1592.1		2,824,555.10	1,352,053.54	130.4		2,138,514.26	949,398.50	56.8	
Wyoming	10,924,502.56	6,040,867.05	1133.5		1,098,360.00	1,098,360.00	182.3		69,921.31	42,256.00	16.7	
Hawaii	966,692,834.36	426,178,703.59	62626.6		343,664.15	79,875,412.15	8079.4		61,715,800.97	24,608,600.07	2620.6	
TOTALS					179,053,956.84	79,875,412.15	8079.4		61,715,800.97	24,608,600.07	2620.6	
* Includes projects reported completed (final vouchers not yet paid) totaling:										Estimated cost \$ 70,576,344.58	Federal aid \$ 30,543,223.16	Miles 2758.8

PUBLIC ROADS

A JOURNAL OF HIGHWAY RESEARCH



UNITED STATES DEPARTMENT OF AGRICULTURE
BUREAU OF PUBLIC ROADS



VOL. 8, NO. 5



JULY, 1927



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PUBLIC ROADS

A JOURNAL OF HIGHWAY RESEARCH

U. S. DEPARTMENT OF AGRICULTURE

BUREAU OF PUBLIC ROADS

CERTIFICATE: By direction of the Secretary of Agriculture, the matter contained herein is published as administrative information and is required for the proper transaction of the public business

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R. E. ROYALL, Editor

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THE FIRST INTERNATIONAL SOIL CONGRESS AND ITS MESSAGE TO THE HIGHWAY ENGINEER

By CHARLES TERZAGHI, Massachusetts Institute of Technology, Research Consultant to the Bureau of Public Roads

DURING the third week of the month of June, Washington, D. C., welcomed more than 200 representatives of soil science from about every part of the civilized world. The meeting was the result of the cooperative efforts of the International Society of Soil Science, the American Society of Agronomy, and the United States Department of Agriculture.

The papers and discussions covered practically the whole field, the application of soil science to highway engineering representing a very small part of it. For this very reason the congress offered a favorable opportunity to survey the various possibilities for future more intimate cooperation, and to revise if necessary some of our conceptions by discussing them with scientists, who approach similar problems from a different angle. However, before dealing with the present situation as revealed by the congress, it may be useful to review briefly the principal points of contact between soil science and highway engineering.

THE SOIL SCIENTIST'S AND THE AMERICAN HIGHWAY ENGINEER'S ATTITUDE TOWARD THE SOIL

Until some 15 years ago road construction was essentially considered a problem in surveying combined with the application of a few simple rules of construction. Very little, if any, attention was paid to the subgrade. Since then various methods have been tried for distinguishing between poor, medium, and good subgrades, and considerable study has been given to the physical properties of the subgrade materials. Nevertheless the tendency exists to consider the road as detached from the landscape, the road consisting of an artificial layer of construction material superimposed on a layer of natural material with a variable resistance. According to this conception, the subgrade represents a second layer of construction material previously deposited by nature, either to the advantage or to the detriment of the engineer. Hence the efforts of the engineer have been exclusively directed toward establishing a simple classification for expressing the relative quality of this second layer.

In contrast to this the soil scientist considers the soil as inseparably connected with geographical and climatic facts. His research extends not only over the surface but also includes the stratification of the ground in a vertical sense (subdivision into A, B, and C horizons). His soil lives, has a definite age, a certain history, and has a considerable number of properties independent of those inherent in the material of which the soil consists. As a matter of fact, most of the papers presented at the congress dealt with the soil not as a material but as a local product which reflects in its properties a great number of different factors, such as the parent material, the age, the drainage conditions, the type of vegetation, the climatic conditions, and many others. Under such circumstances, it is from the soil scientist's view hardly possible to pass reliable judgment on anything concerning a soil without considering it in connection with geomorphological, climatological, and structural facts.

As an example, let us consider what the geomorphologist calls a dissected peneplain—that means an ancient mountain chain which in the course of time was first reduced to almost a plain and subsequently was dissected by a system of rivers and brooks. As a result, the landscape consists of many plateaus with a more or less undulating surface separated from each other by valleys with steeper slopes. Many such peneplains can be found throughout the world, the plateau landscapes of central New England among them. The landscape as faced by the observer represents the result of a series of geological events, viz, the formation of a mountain chain by uplift, the gradual wearing down of the crests and peaks by atmospheric agencies and creep until they practically disappeared, subsequent lifting of the plateau and the formation of the valley system. Hence the landscape reflects in its orographic detail a rather intricate history. The same is true for the anatomy and the morphological features of the soil which, like a living skin, covers the landscape.

As a matter of fact, the skin is the only part of the landscape which actually participated in the modification of the surface topography. It is within and immediately beneath the skin where the process of rock destruction and transportation took place, and the skin represents what was left of the material which in former days was located above it. As a consequence, the skin is everywhere precisely as old as the part of the landscape which it covers, the age being very different in different localities. On the plateau the soil is very old and varies in addition according to the nature of the parent rock located beneath and according to whether it covers the highest or the lowest spots of the area. Along the slopes the soil is very much younger. It is in a state of continuous, comparatively rapid movement down the slopes (creep). Again, at the same age and slope its character (degree of granulation, average moisture content, permeability, etc.) may be different, according to whether it covers a north or a south slope, whether it is located near the edge of the plateau or near the bottom of the valley, on a low crest or between two crests. In brief, the physiographic character of the landscape as revealed by the presence of crests, valleys, and hollows may be considered a very simplified picture of a corresponding variety in age, composition, and structure of the soil skin which covers it.

HIGHWAY ENGINEER HAS IGNORED GEOMORPHOLOGICAL FACTS

Until a short time ago the American highway engineer cared but very little about all these geomorphological details. He located his line, sampled the narrow strip of land which in future will represent the road, trusting that a definite relation may exist between the behavior of the road and the physical properties of the raw material of which the soil at that particular location consists. Yet in constructing his road on top soils with similar physical character his efforts may lead to success or failure, depending on the particular location of his section, which, in turn, governs the hydrogeological conditions. The laboratory investigation

of the raw material extracted from one particular stratum fails to inform him about some vital factors; for instance, the local moisture conditions which, in our assumed case, may be different in spite of the identity of the raw materials. These and similar conditions can only be investigated on the ground by a competent observer; but until quite recently no attempt was made by American highway engineers to organize a field service for investigating and recording the geomorphological facts which have a bearing on highway construction.

Here is the point from which the Russian engineers started. The Russian attempts to establish scientific principles for subgrade investigations originated at a very much later date than the American ones. The American experiences were at their disposal. In addition to this it was in Russia where, for the first time, a systematic study was made of the intimate relations which exist between the soil, the physiographic character of the landscape, and the climatic conditions. (V. V. Dokoutchaev, 1896-1900, and his distinguished disciples N. M. Sibirtzev and K. D. Glinka.) Hence from the very beginning the Russian highway engineers realized the weaknesses inherent in the American method.

THE RUSSIAN METHOD OF SUBGRADE INVESTIGATION

The Russian attitude toward the subgrade problem was recently discussed in a paper presented by N. Y. Prokhorov to the U. S. S. R. Academy of Sciences. (1927, Russian Pedological Investigations, XII.) This paper contains a rather severe criticism of the American methods of subgrade investigation and gives a very interesting review of what has been done in Russia to improve upon them. As a result of some preliminary work carried out in 1923, a new department was created in 1924 under the title "Highway research bureau of the central department of local transport of the people's commissariat of ways and communications," including subdivisions for laboratory investigations, engineering, economic studies, soil science, and field observations (highway research parties). The program of the organization clearly expresses the tendency to study the subgrade in connection with the landscape and the climate as a natural body and not a "chance association of pulverized matter." Soil tests made in the laboratory similar to those practiced in the United States form part of the program but at the same time are considered second in importance. The main efforts were directed toward establishing standard methods for expressing the results of field observations on road maps which show not only the character of the narrow strip of land to be occupied by the road but also the adjoining belt, together with its hydrogeological characteristics and the stratification of the soil from the surface down to the parent material. (See, for instance, P. K. Khmyznikov, *Methods for Mapping Soil Formations in Connection with Highway Construction*; V. S. Makarevich, *The Road as an Object of Study for the Geographer*; *Geomorphological and Peto-Botanical Maps for Highway Purposes*, for the Saratov Colony near Leningrad, plotted by P. K. Khmyznikov.)

During the short period of its existence the highway research bureau has established highway experimental stations north of the Caucasus, in Kharkov (Ukraine), and in the Far East. Others are planned in Moscow, Karelia, and Novosibirsk. Field studies are made on

experimental highways and on existing roads which have proved to be either outstanding failures or successes (for instance, of a remarkably well preserved road 200 miles long, between the Reynovo station on the river Amur and the Aldan gold fields). Several new road projects were handled in intimate cooperation with some of the most prominent soil scientists with special consideration of the possibilities for improving the quality of the subgrade by admixture of foreign material to be extracted from "subgrade quarries." (A. E. Nazarenko, *Admixture of Subgrade in the Construction of Common Roads*.) V. V. Nikitin and B. V. Gervais are engaged in a study of the methods for investigating and recording climatological facts for highway purposes. Thus, in spite of the brief existence of the highway research bureau, it is probable that there has been accumulated in the Russian publications sufficient material to enable us to pass judgment on how far the Russian engineers have succeeded in turning their intentions into practice.

A COMPROMISE BETWEEN THE AMERICAN AND THE RUSSIAN METHOD DESIRABLE

There is no denying that the Russian method of dealing with the subgrade problem has much in its favor. A laboratory test, no matter how perfect it is, tells only part of the story. The other, and probably more important, part has to be learned by direct observation in the field. As a matter of fact, by its own initiative and without any knowledge of the Russian precedent, the Bureau of Public Roads has included in its program of the proposed condition survey the cooperation of both a soil scientist and a geologist. The first is expected to furnish information on the soil profile and the second on the geological and hydrogeological conditions. Yet, for the time being, this measure merely expresses the fact that the presence of a gap is felt. The question of how to fill the gap is still open. It remains to determine what to observe and how to condense the results of the observations into a reasonable amount of data. In connection with these questions a thorough study of the Russian experiences undoubtedly may prove to be both very stimulating and very helpful.

Yet even the Russian procedure is still in an initial stage of development. For this reason it was particularly profitable to get at the soil congress first-hand information concerning where we stand at present and to investigate how best to take advantage of the recent developments in that field. In what follows an attempt is made to present the results of this investigation and to comment upon the future possibilities of keeping in contact with the different branches of soil science.

AGREEMENT NOT YET REACHED ON SYSTEM OF SOIL CLASSIFICATION

Soil classification for foundation purposes was discussed in a paper presented by the writer. It should, however, be stated in advance that this classification has nothing in common with any of the soil classifications worked out by the soil scientists. Foundations are established on or within the C horizon which, by its very nature, merely represents "an accumulation of pulverized material." Hence the classification for foundation purposes is based exclusively upon mechanical properties, as is the classification of any other kind of inorganic construction material. In contrast to this the soil scientist dwells upon the properties of the

top soil and of the B horizon, which are in a state of continuous change and which, in addition, represent the result of a gradual development.

In constructing a road we depend upon the resistance of the subgrade. Hence the mechanical properties (compressibility, elasticity, consistency, etc.) of the layer which supports the road are factors of importance and the classification of the raw material of which this layer consists ought to be based on purely mechanical characteristics. Yet at the same time the prevalent moisture conditions and the subdrainage depend on the soil profile and on the position of the area within the landscape. For properly investigating and expressing these facts we depend upon the methods and the experience of the soil scientist. Therefore we are vitally interested to learn to what extent the soil scientist has succeeded in developing and standardizing his methods in that particular field.

From what was presented at the congress it seems evident that the original Russian method of classifying the soils according to the factors instrumental in their history has been superseded by a classification based upon properties inherent in the soil. For several years this new method has been strongly advocated by C. F. Marbut, of the United States Department of Agriculture, and to-day its merits seem even to be recognized by the representatives of the Russian school. At the same time it is obvious that no final agreement has yet been reached concerning the properties on which such classification should be based. Charles F. Shaw, in his classification of California soils, considered the following properties: (1) Origin (order), (2) lime content (class), (3) parent rock (division), (4) character of hardpan layer (family), (5) age (stage), (6) color (group), (7) various other properties (series). S. A. Zakharov bases his classification on the following properties: (1) Color, (2) structure, (3) porosity and compactness, (4) depth, (5) character of Neo formations (concretions and holes), (6) soil skeleton, and (7) soil profile. J. Hendrick and G. Newlands, of Aberdeen, Scotland, give much importance to the mineralogical composition of the "fine sand fraction," which, according to their opinion and experience, is closely related to the composition of the finest soil constituents. M. M. McCool, Michigan, calls our attention to the importance of the percentage of colloids present in the soil, yet admits that the properties of two soils with an equal colloid content may be very different depending on the nature of the colloids. A. F. Lebedev, from Rostov on the Don, claims that soil mapping based on the determination of soil types is "indefinite and incomplete, the point of view of the investigator prevailing." He proposes to prepare separate maps showing the distribution of the moisture content or mechanical composition, reaction facts, air content, and other physical facts. The value of such maps would remain unaffected by a change in our methods of soil classification and broaden the field of possible application of the results of surveys.

HIGHWAY ENGINEERS MUST SECURE ASSISTANCE OF SOIL SCIENTISTS

As a result of the lack of agreement combined with a rather general confusion concerning the terminology of soil science, we are still very far from a universally accepted method of soil survey. But even within the same State, for instance, within the boundaries of the United States, it is very difficult for the highway engi-

neer to take advantage of the information accumulated in the records of the Bureau of Soils. The soil classification of the bureau is based on all the properties of the soils irrespective of whether or not they have an effect on their quality as subgrades. As a consequence, the soil maps show an alarmingly great variety of soils, hardly any soil type occurring beyond the limits of a rather restricted area. Many of these soils may, from the highway engineer's point of view, be perfectly identical. On the other hand, the same soil type may be rather different in its properties as a subgrade according to its location within the landscape and according to the hydrogeological conditions existing in its vicinity.

The only method for remedying this situation seems to consist in securing the services of soil scientists who, in addition to their expert knowledge in the field of soil science, are thoroughly familiar with the requirements of highway engineering. Because of their training in soil science, they would be in a position to take full advantage of the achievements of science in this particular field. On the other hand, through their permanent contact with the highway engineer they would learn to discriminate between those properties of the soils which have a bearing on road construction and those which can be considered as unimportant. The activities of such highway soil scientists would ultimately lead to a soil morphology similar to that which serves as a basis for general soil surveys yet simplified and modified for highway purposes. The modification of the basic principles would be associated with a modified method of soil mapping specially adapted for highway purposes. The results of the soil tests would merely serve as a check on the information obtained by the soil scientists in the field.

That is precisely the way in which things have developed in Russia since 1924, and, according to N. J. Prokhorov's statement, both the field methods and the methods for soil mapping for highway purposes have been brought to a certain degree of perfection. It is obvious that the task involved in developing such methods is a job neither for the average highway engineer nor for the average soil scientist, because the existing methods must be adapted to an altogether new field of application. Such work requires a keen gift of observation and independent judgement combined with an unusually broad knowledge in the field of geomorphology, hydrography, and soil science. Yet, since we find some of the very best names connected with the Russian venture (Glinka, Gedroiz, Krynin, and many others), much labor and disappointment could be avoided by a more intimate contact with the work of the Russian investigators.

An exceptional position in the soil system is occupied by the soils of almost purely organic origin (peat, muck, and others). In contrast to the other soils they are almost international in their character, each type of peat deposit occurring in widely separated parts of the globe. This fact was brought out by A. P. Dachnowski of the United States Department of Agriculture. He distinguishes between four fundamental types of peat and develops the basic principle for the classification of soils of organic origin. The bearing of Doctor Dachnowski's investigations on highway engineering has already been suggested in a paper, published in the February issue of PUBLIC ROADS.¹ Because of the

¹ Fill Settlement in Peat Marshes, by V. R. Burton, PUBLIC ROADS, vol. 7, No. 12, Feb. 1927.

uniform character of peat deposits, it will ultimately be possible to utilize the experience gained in one locality in similar construction work to be executed in many other places. Hugo Osvald, of Jönköping in Sweden, showed very instructive cross sections of two peat bogs in Maine.

NEW DEVELOPMENTS IN SOIL PHYSICS PRESENTED

In the field of soil physics one can not escape the depressing feeling that much time and efforts is wasted because the investigators are quite often unfamiliar with some—if not with all—of the physical principles involved in their investigations. Compression tests are made without paying any attention to the laws of the resistance of materials. Authors speak about "permeability" as if the permeability of a soil could be expressed by a single numerical value, and no distinction is made according to whether the permeable system has a perfectly rigid or a deformable structure. Moisture equivalent and plasticity are discussed as if they had no connection with the common laws of physics. The same is true for what concerns the volume changes and the capillary phenomena. Yet there are some noteworthy exceptions to this rule, which is proved, for instance, by the high standard of the work performed at the Rothamsted Agricultural Experiment Station in Harpenden, England.

The efforts of the Bureau of Public Roads formed part of the subject of a paper presented to the congress by the writer. They essentially consist in a systematic study of the purely mechanical properties (compressibility, elasticity, consistency, permeability, and the like) of the raw material of the soils and in an attempt to correlate all these properties with the results of simple and rapid routine tests. As a result of this attempt, the bureau expects to establish what may be called a calibration of the simple routine tests—a set of charts by means of which one could determine without any further effort all the different features of the mechanical character of the soil. Since, for each example, the routine tests do not require more than about one hour of actual work, the calibration may ultimately represent the most efficient means for rapidly investigating any given material, regardless of the purpose of the investigation. It is clearly recognized that the information thus obtained will form a fraction only of the information required for judging the character of a subgrade. Yet, within its limits, the method will fully serve its purpose, securing a maximum of information with a minimum effort. B. A. Keen and J. R. H. Coutts, of the Rothamsted Experimental Station, presented valuable data on the effect of the organic constituents on the physical properties of the soils. By treating their samples with hydrogen peroxyd, they succeeded in almost completely removing the organic constituent without materially changing the character of the inorganic fraction. By using this method they removed the organic constituents from 40 different samples and investigated the physical properties of both the treated and the untreated samples. The investigations included the hygroscopic water content, the water content at 50 per cent relative humidity, the volume change due to drying, and the "sticky limit." The test results showed among other things that the removal of the organic material caused a lowering of the "sticky limit."

L. Smolik concluded from some of his experiments that the fresh soil contains a higher percentage of very fine material (particles smaller than 0.002 millimeters) than the same soil after being dried at room temperature.

One of the most interesting physical facts brought to the attention of the congress concerns the effect of electro-endosmose on the coefficient of friction between wet soil and steel. By sending an electric current through the soil the coefficient of friction between the negative electrode and the soil decreases, the amount of the decrease ranging between zero and 80 per cent of its original value, and the rate of decrease depending on the intensity of the current and the water content of the clay. According to the diagram exhibited by B. A. Keen, of the Rothamsted Experimental Station, a decrease of 80 per cent in friction value corresponds to a water content of 20 to 23 per cent and to a voltage of about 60. A further increase of the voltage has but little effect on the friction. At the surface of contact between the soil and the positive electrode the friction shows a corresponding increase. According to Prof. J. Schetelig, of Oslo, Norway, tests were made in Norway which seem to show that the sudden decrease in internal friction associated with the sliding of certain blue glacial clays is also an electric phenomenon caused by a change in the state of polarization at the boundary between the mica flakes and the water. These discoveries certainly will have sooner or later an important bearing on every branch of engineering dealing with soils.

METHODS FOR INVESTIGATING DRAINAGE PROPERTIES IN CONTROVERSIAL STATE

The methods for investigating the drainage properties of the soils are, as it seems, still in a very controversial state. For the time being five different methods are used for estimating the drainage properties, viz, (1) mechanical analysis (Kopecky, Fauser) (2) hygroscopicity (Rodewald-Mitscherlich), (3) determination of the specific surface of the soil (Zunker), (4) direct determination of the permeability of the undisturbed soil (Freckmann, Jaenert, Schroeder), and (5) from observations while lowering the groundwater level. There are claims that every one of these methods is successful, yet the results obtained by means of the various methods vary within wide limits. This, after all, is not surprising, because the mere presence of drains changes the nature and the permeability of the adjoining soil in a way which can certainly not be predicted from the result of a permeability test performed in the laboratory. Hence it was enjoyable to learn at the congress about the increasing attention paid to actual field tests. W. J. Schlick, of the Iowa State College; F. O. Bartel, of the United States Department of Agriculture; and R. Janota from Prague, Czechoslovakia, reported on the results of rather extensive tests made in the field for the purpose of investigating the relations which exist between the permeability of the underground (according to the results of laboratory tests), the position of the water table, and the run-off. Janota supplemented his data with the results of observations concerning the changes which occurred in the soil while draining it.

C. W. Botkin, of the New Mexico college, investigated in the laboratory the effect of alkali and fertilizers

on the permeability of Gila clay and Gila clay loam. He found that all those ingredients which cause flocculation also increase the permeability, while the deflocculating agents made the soil more impermeable.

Some interesting observations concerning the importance of irrigation by condensation were reported by A. F. Levedoff. The amount of water drawn by the top soil of the semiarid regions of southern Russia out of the air during the night by condensation was found to be not less than 70 millimeters per year (about 3 inches). L. B. Olmstead, of the Department of Agriculture, presented thermograph records showing the variations in temperature of the air and of the soil at a depth of 1, 6, and 18 inches, on a bare, uncultivated plot at Arlington, Va.

NEW EVIDENCE THAT QUANTITY OF COLLOIDS DOES NOT DETERMINE CHARACTER OF SOIL

There are few facts in soil science which acted stronger on the imagination of foundation and highway engineers than did the colloid content of the soils. It was apparently believed that the "soil colloid" is a specific and well-defined substance with very unusual properties, and that the quantity of this substance present in the soil alone is sufficient to determine the character of the soil. The soil congress merely brought some new and rather convincing evidence that the soil colloids may differ from each other as widely as the properties of the soils. Thus, Saute Mattson, of Sweden, reported on the results of tests with seven different soil colloids whose ratio of SiO_2 to $\text{Al}_2\text{O}_3 + \text{Fe}_2\text{O}_3$ ranged between the limits 0.55 and 3.82. The red and yellow colloids with a low ratio were found to absorb considerably more anions from hydrochloric and sulphuric acid than did the gray colored ones with a high ratio. The same investigator, by treating the colloidal fraction of a soil with AlCl_3 , succeeded in splitting the fraction up into two parts with considerably different ratios of SiO_2 to $\text{Al}_2\text{O}_3 + \text{Fe}_2\text{O}_3$.

M. M. McCool, of Michigan, observed a remarkable difference in color and tenacity between those colloids which accumulated at the base and those colloids which formed the top part of the sediment in the tubes of the supercentrifuge, although both colloids came out of the same soil. Some soils with 20 per cent colloids extracted from a B horizon were found to be far more tenacious than others with 40 to 70 per cent. These few examples may be sufficient to demonstrate once more that the quantity of colloids present in the soil is far away from determining the character of the soil. In order to learn something about the soil it is also necessary to determine the type of the colloids contained in the soil and their state of adsorptive saturation. Under these conditions, soil chemistry may provide us sooner or later with some useful supplementary routine tests, but the center of gravity of our laboratory methods will continue to reside in the physical soil testing.

CERTAIN SOIL INGREDIENTS ATTACK CONCRETE

The only aspect of soil chemistry which is of vital interest to the highway engineer concerns the effect of certain soils on concrete. G. Wiegner and H. Gessner, from Zürich, reported on the results of very extensive investigations made by the "commission for the investigation of the deterioration of concrete pipes in reclama-

mation districts." In certain parts of Switzerland the deterioration of cement pipes occurred on an alarming scale. Subsequent investigations showed that the cement pipes were damaged even in localities where no detrimental effect of soil conditions on the pipes was expected. The destruction was found to be due to acidic soil reaction, to the presence of carbonic acid in the ground water, the presence of a high content of gypsum (certain peat deposits), or of a high content of magnesium (certain soils rich in lime). Great density of the concrete or mortar is apt to retard but not to prevent the ultimate destruction. Based on the results of the investigations, the commission considers it necessary to request an analysis of the subsoil in connection with any reclamation project which involves a considerable investment in cement or concrete drainage pipes. It may be that concrete road surfaces, too, could be damaged in contact with soils whose ingredients attack the cement. If this be the case, the experiences incorporated in the reports of the Swiss commission may be of great value.

The Russian delegates exhibited a most instructive set of complete soil profiles showing the variation with varying depth of the hygroscopic water content, the humus content, the content in Fe_2O_3 , Al_2O_3 , SiO_2 , the percentage of particles smaller than 0.01 millimeter present in the soil, and the base adsorption in calcium equivalent. Although such profiles have no direct bearing on highway problems, they demonstrate the necessity of investigating not only the topsoil but the whole soil profile.

METHODS OF TESTING DISCUSSED

Among the discussions on the methods of testing, the methods for performing the mechanical analysis maintained their prominent place. Wiegner and Gessner, of Zürich, exhibited a very ingenious device whose operation merely requires the dispersion of the soil by shaking the apparatus, the turning on of a light, and the setting in motion of a revolving film. One or two days later the grain size curve can be taken from the drum. (Price approximately \$150.) The Rothamsted Agricultural Experiment Station showed another apparatus by means of which the mechanical composition of the soil can be determined from the density of samples withdrawn from the sedimentation tank with a pipette.

Yet it still remains doubtful to what extent the results of the analysis can be relied upon and how the samples should be treated prior to the analysis so as to secure perfect dispersion. Charles F. Shaw and E. V. Winterer, of the University of California, call attention to a fundamental error associated with every type of mechanical analysis based on the velocity of settling. On account of their electric charge, the particles do not move vertically downward but diagonally toward the wall, and the process of sedimentation proceeds simultaneously with a flocculation near the walls of the vessel. Hence, he raises the question whether it is possible at all to determine the percentage of particles smaller than 0.002 millimeter with a reasonable degree of accuracy by any method based on the rate of settling. G. J. Bouyoucos, of the Michigan Agricultural Experiment Station, goes even further than that. He states that the colloidal content of soils as indicated by the hydrometer method is very much higher than the one determined by mechanical

analysis, to such an extent that a new and radical textural reclassification of the soils is needed. M. M. McCool, of Michigan, too, points out that the results obtained by the standard method of mechanical analysis do not represent the true amounts of fine sand, silt, and clay. Finally, considering the amount of labor required to prepare the samples for the mechanical analysis and the uncertainty of the effect of such treatment on the soil, the analysis remains a rather unattractive test.

Yet even discarding the mechanical analysis from the routine program, the necessity exists to separate the coarse sand fraction from the finer material. For this kind of work a very convenient tool has been invented by M. Koehn and R. Albert. It is based on the principle of separation of the particles according to grain size by an upward current of water. The current is produced by a centrifugal pump coupled with a motor which, by means of a special type of coupling, accurately regulates the speed of the flow. Thus the separation of the soil into fine and coarse constituents proceeds automatically.

How to determine and express the color of the soils is another rather important problem. At the Bureau of Public Roads the color standards and color nomenclature by Robert Ridgeway are used. This method is neither accurate nor convenient, and the names of the colors mean nothing except to those who are thoroughly familiar with both the colors and the standard. As a result of the labors of the soil colors standards committee of the American Soil Survey Association, a factorial system of color description will soon be available for general use. According to this system, the different colors can be produced by the rapid rotation of colored disks, the value of the color being expressed by the percentage of space occupied by the elementary colors on the revolving disk. The Russian soil experts use W. Ostwald's system, which is essentially based on the ratio between color, black and white. T. M. Bushnell points out that the soil colors do not include more than 6 per cent of all the existing colors.

NEW DYNAMOMETER MAY BE USEFUL TO HIGHWAY ENGINEER

A very important innovation consists in the Rothamsted self-registering dynamometer. This dynamometer serves for measuring the variations in the resistance of the ground against the cutting action of the plow. It essentially consists of a curved brass tube filled with oil, the tube being inserted between the tractor and the plow. The strain produced in the tube by the pull of the tractor is recorded on a traveling strip of celluloid with a magnification of 20 to 1. By traveling with a plow back and forth over a field, the data were obtained for plotting the curves of equal resistance. The results furnished by such operations disclosed the important fact that the resistance of the soil varies within wide limits, from point to point, even on fields which according to their appearance seemed to be perfectly uniform. Repeating the test at different times of the year under different moisture conditions, practically the same curves of equal resistance were obtained, although the absolute values of the resistance were very different.

This reminds us of the fact that in a vertical sense, too, the character of the strata varies from point to point, and the data obtained by the Rothamsted tests give us some conception of the importance of the errors

due to the limited number of samples taken in the field in connection with road surveys. It may be that the Rothamsted dynamometer could be used in connection with the soil survey for road-construction purposes. A dynamometer record would cover the full length of the road. It would give us reliable information concerning the variations in the resistance of the ground. It would disclose the presence of weak or excessively humid spots and would serve as a guide for properly selecting the soil samples. Yet the application of the dynamometer method would obviously be confined to surveys on level or very slightly inclined ground outside of forested districts.

Finally, the tools for sampling should be mentioned. A. P. Dachnowski, of the Department of Agriculture, displayed the tools which he uses in his investigation of peat deposits. His peat sampler furnishes undisturbed cores of peat 6 inches long and with a diameter of three-fourths of an inch. This tool certainly should be used by every highway engineer who has to deal with peat deposits. The Missouri Agricultural Experiment Station exhibited a tool for obtaining cores 16 inches long, with a diameter of 6 inches, from the bottom of pits. It consists of a core barrel which closely fits the specimen and of a rotating shell whose lower edge is provided with three cutting knives. By rotating the shell, a groove is cut all around the specimen. As soon as the top of the core arrives at the upper edge of the core barrel, the shell is withdrawn and the core remains protected by the barrel and surrounded by an annular space. H. H. Musselman, of the Michigan State College, exhibited a sampler for taking marl cores.

CONCLUSIONS

The first international soil congress gave to the highway engineer a chance to survey the possibilities for a closer cooperation with the Bureau of Soils and similar institutions. From the papers presented at the congress it became evident that the methods used by the soil scientist require a rather thorough modification to be applicable to highway engineering. At the same time it was learned that the Russian highway engineers, in close cooperation with their soil experts, have made what seems to be a successful effort to work out the required modifications, thus making it possible to utilize the results of soil research to full advantage. Hence, it seems desirable to make a careful survey of the present practice of the Russian methods of subgrade investigation.

Among the different branches of soil science, soil genesis, soil morphology, and the methods of soil survey undoubtedly have the most direct bearing on highway engineering. The results obtained in the field of soil physics are helpful inasmuch as they deepen our insight into the mechanics of soil behavior. Soil chemistry may, sooner or later, furnish some useful supplementary routine tests. Yet, as a whole, the points of contact between soil chemistry and highway engineering are of minor importance.

Valuable data were presented concerning the detrimental effect of certain soil solutions on cement pipes and on concrete.

Among the new tools which were developed during the last few years the following seem to be of interest to the highway engineer: A machine for automatically extracting the sand fraction out of the soil, the Rothamsted self-registering dynamometer, and various tools for obtaining undisturbed samples of soil.

ANALYSIS OF CONCRETE ARCHES

PART 2.—ANALYSIS OF UNSYMMETRICAL ARCHES

By W. P. LINTON, Highway Bridge Engineer, and C. D. GEISLER, Associate Highway Bridge Engineer, United States Bureau of Public Roads

PART 1 of this article, published in the preceding issue of PUBLIC ROADS, presented the derivation of formulas for arch design and the calculations for the design of a symmetrical arch using the forms which have been developed to lessen the labor and chances of error in arch calculations. It will be assumed that the reader is familiar with the preceding article, and an example will be worked out for an unsymmetrical arch, explaining only such steps as have not already been discussed.

FORMULAS FOR UNSYMMETRICAL ARCHES

The following formulas for unsymmetrical arches were derived. They are applicable to either symmetrical or unsymmetrical arches, but for symmetrical arches it is much easier to use them in the simplified form previously explained.

$$V_o = \frac{1}{2} \sum_a^l (z-k) \Delta \left(z - \frac{\sum z \Delta}{\sum \Delta} \right) \quad G = \frac{1}{2} \sum_a^l (z-k) \Delta \left(y - \frac{\sum y \Delta}{\sum \Delta} \right)$$

$$F = \frac{BG}{C}$$

$$H_o = \frac{V_o B - \frac{1}{2} \sum_a^l (z-k) \Delta \left(y - \frac{\sum y \Delta}{\sum \Delta} \right)}{C}$$

$$M_o = \frac{dx}{\sum \Delta} \frac{1}{2} \sum_a^l (z-k) \Delta + H_o \frac{\sum y \Delta}{\sum \Delta} - V_o \frac{dx \sum z \Delta}{2 \sum \Delta}$$

$$M_x = M_o + [V_o z - (z-k)] \frac{dx}{2} - H_o y$$

$$V_t = \frac{2r}{(dx)^2 + 20} \frac{G}{C} - \frac{BG}{F - \frac{BG}{C}} - eE$$

$$H_t = \frac{V_t B + 20 e t E}{C}$$

$$M_t = -H_t \left(y - \frac{\sum y \Delta}{\sum \Delta} \right) + V_t \frac{dx}{2} \left(z - \frac{\sum z \Delta}{\sum \Delta} \right)$$

$$N_x = H \cos \phi + V_x \sin \phi$$

$$f_c = \frac{N}{A} \pm M \frac{h}{2I}$$

$$B = \frac{1}{2} \sum z \Delta \left(y - \frac{\sum y \Delta}{\sum \Delta} \right)$$

$$C = \frac{1}{dx} \sum y \Delta \left(y - \frac{\sum y \Delta}{\sum \Delta} \right) + \sum \frac{\cos \phi}{A}$$

$$F = \frac{1}{2} \sum z \Delta \left(z - \frac{\sum z \Delta}{\sum \Delta} \right)$$

$$G = \frac{1}{dx} \sum z \Delta \left(y - \frac{\sum y \Delta}{\sum \Delta} \right)$$

CALCULATIONS FOR AN UNSYMMETRICAL ARCH

An unsymmetrical arch of the dimensions shown in Figure 1 is to be designed to carry a live load of 125 pounds per square foot and to withstand a rise of temperature of 30° F. or a fall of 40°.

The general method of procedure is identical with that described for symmetrical arches. The dimensions of the arch ring and reinforcement may be arrived at by any of the methods in general use, and a drawing similar to Figure 1 is prepared on a scale of 1 inch to 3 feet. For unsymmetrical arches it is necessary to draw the entire arch ring.

Computations are entered in 12 tables, which can be conveniently arranged on six sheets of letter-size paper grouped as presented here. Bold-face type is used to indicate the results of calculations, and plus and minus signs which are the same for all arches and may be printed on the blank forms. Since the arch is unsymmetrical, the tables must be filled out for the entire arch in every case. As in the preceding example for a symmetrical arch, coefficients will be determined for a load of unity at the various points, and these coefficients will be applied to the dead and live load to determine the stresses.

Table 1. Computations for Δ .—This table is filled out in the same way as Table 1 for the symmetrical arch and requires no additional explanation.

Table 2. Computations for $\frac{1}{2} \sum_a^l (z-k) \Delta \left(y - \frac{\sum y \Delta}{\sum \Delta} \right)$.—

The value of this expression, which appears in the formula for V_o , was not found for the symmetrical arch, since it is multiplied by G , which in the case of the symmetrical arch is zero. The value of y for each of the 20 points is scaled and recorded in column 12. The value of $y \Delta$, column 13, is the product of the corresponding values of y and Δ in columns 12 and 11. The sum of column 13 should be divided by the sum of column 11 and the value of $\frac{\sum y \Delta}{\sum \Delta}$ is recorded in the place provided on the same sheet with Tables 1 and 2.

Column 14 is computed by subtracting $\frac{\sum y \Delta}{\sum \Delta}$ from each value of y . Column 15 is computed by multiplying each term in column 14 by the corresponding term in column 11. If the work in columns 13, 14, and 15 is correct, the sum of column 15 will be zero.

Columns 16 and 17 are computed in exactly the same manner as explained for columns 36 and 37 of Table 4 for symmetrical arches. The figure opposite point 20 in column 16 is always zero. To this add the figure opposite point 20 in column 15 with its proper algebraic sign and write the result in the next space in column 16. To this add algebraically the next figure in column 15 which is opposite point 19, and set down the sum in column 16 opposite point 18. Then add the next figure in column 15 and continue this process to the top of the page. Since the sum of column 15 is zero, the top figure in column 16, opposite point 1, will be numerically equal to the top figure in column 15.

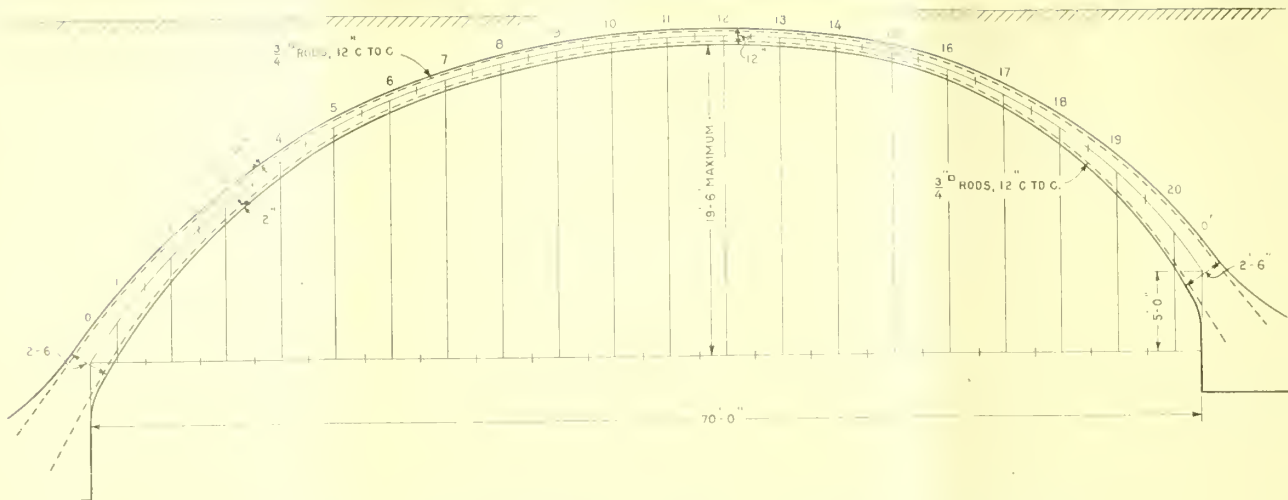


FIG. 1.—Sketch of Arch Ring Showing Principal Dimensions and Load Points

TABLE 1.—Computations for Δ

	1	2	3	4	5	6	7	8	9
Points	h	h^3	I_c	$\frac{h}{2}$	$\left(\frac{h}{2} - d'\right)$	$\left(\frac{h}{2} - d'\right)^2$	I_s	$I = I_c + I_s$	
0	2.50	15.625	1.302	1.25	1.08	1.166	0.136	1.438	
1	2.32	12.487	1.041	1.16	.99	.980	.115	1.156	
2	1.92	7.078	.590	.96	.79	.624	.073	.663	
3	1.60	4.096	.341	.80	.63	.397	.046	.387	
4	1.34	2.406	.200	.67	.50	.250	.029	.229	
5	1.20	1.728	.144	.60	.43	.185	.022	.166	
6	1.10	1.331	.111	.55	.38	.144	.017	.128	
7	1.08	1.260	.105	.54	.37	.137	.016	.121	
8	1.06	1.191	.099	.53	.36	.130	.015	.114	
9	1.04	1.125	.094	.52	.35	.122	.014	.108	
10	1.02	1.061	.088	.51	.34	.116	.014	.102	
11	1.00	1.000	.083	.50	.33	.109	.013	.096	
12	1.00	1.000	.083	.50	.33	.109	.013	.096	
13	1.00	1.000	.083	.50	.33	.109	.013	.096	
14	1.02	1.061	.088	.51	.34	.116	.014	.102	
15	1.06	1.091	.099	.53	.36	.130	.015	.114	
16	1.10	1.331	.111	.55	.38	.144	.017	.128	
17	1.20	1.728	.144	.60	.43	.185	.022	.166	
18	1.40	2.744	.229	.70	.53	.281	.033	.262	
19	1.84	6.230	.519	.92	.75	.562	.066	.585	
20	2.26	11.543	.962	1.13	.96	.922	.108	1.070	
0'	2.50	15.625	1.302	1.25	1.08	1.166	.136	1.438	
$\Sigma \Delta$ 483.16									
$\Sigma y\Delta$ 8455.7136									
0.0000									

TABLE 3.—Computations for $\frac{1}{2} \sum_a^l (z-k)\Delta \left(z - \frac{\sum z\Delta}{\sum \Delta} \right)$

18	19	20	21	22	23	24
Points	z	$z\Delta$	$z - \frac{\sum z\Delta}{\sum \Delta}$	$\Delta \left(z - \frac{\sum z\Delta}{\sum \Delta} \right)$	$\sum_a^l \left(z - \frac{\sum z\Delta}{\sum \Delta} \right)$	$\frac{1}{2} \sum_a^l (z-k)\Delta \times \left(z - \frac{\sum z\Delta}{\sum \Delta} \right)$
0	0					
1	1	5.23	-19.606714	-102.5431	+102.5432	19,309.05
2	2	23.97	-17.606714	-140.6776	+243.2208	19,206.51
3	3	60.10	-15.606714	-187.5927	+430.8135	18,963.29
4	4	125.30	-13.606714	-243.5602	+674.3737	18,532.48
5	5	213.03	-11.606714	-274.7309	+949.1046	17,858.10
6	6	322.30	-9.606714	-281.4767	+1230.5813	16,909.00
7	7	392.21	-7.606714	-229.4946	+1460.0759	15,678.42
8	8	473.70	-5.606714	-177.0600	+1637.1359	14,218.34
9	9	558.79	-3.606714	-118.5527	+1755.6886	12,581.21
10	10	651.89	-1.606714	-55.1246	+1810.8150	10,825.52
11	11	765.66	+0.393286	+14.3392	+1796.4758	9,014.70
12	12	838.58	+2.393286	+87.2592	+1709.2166	7,218.23
13	13	914.00	+4.393286	+160.6185	+1548.5981	5,509.01
14	14	937.17	+6.393286	+221.9110	+1326.6871	3,960.41
15	15	915.82	+8.393286	+265.0600	+1061.6271	2,633.72
16	16	896.21	+10.393286	+300.4699	+761.1572	1,572.10
17	17	771.21	+12.393286	+289.6311	+471.5261	810.94
18	18	567.70	+14.393286	+233.4591	+238.0670	339.41
19	19	308.58	+16.393286	+135.7200	+101.3470	101.35
20	20	214.89	+18.393286	+101.3470	0.0000	0.00
0'	40					
		$\sum z\Delta=9956.34$		$\sum=0.0000$		$\sum=+4,695.93 + 5,310.19$

TABLE 4.—Computations for B, C, F , and G

25	26	27	28	29
$z\Delta \left(y - \frac{\sum y\Delta}{\sum \Delta} \right)$	$y\Delta \left(y - \frac{\sum y\Delta}{\sum \Delta} \right)$	$\cos \phi$	$\cos \phi$ A	$z\Delta \left(z - \frac{\sum z\Delta}{\sum \Delta} \right)$
			0.549	
			.575	.248 - 102.54
			.666	.347 - 422.03
			.764	.477 - 937.96
			.844	.630 - 1,704.92
			.896	.746 - 2,472.58
			.940	.854 - 3,093.24
			.962	.890 - 3,083.43
			.975	.920 - 2,655.90
			.987	.948 - 2,015.40
			.996	.976 - 1,047.40
			.999	.999 - 301.12
			1.000	1.000 - 2,006.96
			.998	.998 - 4,015.46
			.990	.970 - 5,991.60
			.975	.920 - 7,686.74
			.948	.862 - 9,314.57
			.903	.752 - 9,557.83
			.812	.580 - 8,171.07
			.718	.390 - 5,058.64
			.605	.268 - 3,952.53
			.558	
				$\sum=14.775 + 38,618.12$

$$\frac{z\Delta}{\sum \Delta} = 20.606,714$$

$$C = \frac{1}{dx} \sum_a^l y\Delta \left(y - \frac{\sum y\Delta}{\sum \Delta} \right) + \sum_a^l \frac{\cos \phi}{A} = 1,531.97$$

$$F = \frac{1}{2} \sum_a^l z\Delta \left(z - \frac{\sum z\Delta}{\sum \Delta} \right) = 19,309.06$$

$$G = .875797$$

$$\frac{BG}{C} = 2,056.34$$

$$B = \frac{1}{2} \sum_a^l z\Delta \left(y - \frac{\sum y\Delta}{\sum \Delta} \right) = \frac{1}{2} \sum_a^l \text{Col. 25} = 2,347.965$$

$$G = \frac{1}{dx} \sum_a^l z\Delta \left(y - \frac{\sum y\Delta}{\sum \Delta} \right) = 1,341.6943$$

$$F - \frac{BG}{C} = 17,252.72$$

$$\frac{1}{C} = 0.000,652,75$$

$$\frac{1}{F - \frac{BG}{C}} = 0.000,057,961,9$$

they are independent of the loading. The values in column 25 are computed by multiplying each term in column 15 by the corresponding value of z in column 19. B is equal to one-half the sum of column 25 and G is equal to $\frac{1}{dx}$ times the sum of column 25. Their values should be set down in the places provided on the sheet containing Tables 3 and 4.

Column 26 is computed by multiplying each term in column 15 by the corresponding value of y in column 12. $\cos \phi$ (see fig. 3, Pt. I) in column 27 might be computed from the formula $\cos \phi = \frac{dx}{ds}$, but since $\cos \phi$ varies so little for small angles more accurate results will be obtained by computing $\sin \phi$ from dimensions scaled from the drawing and then taking the corresponding values of $\cos \phi$ from a table of trigonometric functions. This may be done on a supplementary sheet of paper. It is not necessary to find the values of ϕ as only $\cos \phi$ is wanted now, but the values of $\sin \phi$ should be kept because some of them will be wanted later.

Column 28 is computed by dividing each term in column 27 by A , the area of the arch ring section, which for practical purposes may be taken as equal to h (column 2).

C is equal to the sum of column 28 plus $\frac{1}{dx}$ times the sum of column 26. Its value should be set down in the space provided below Table 4.

Column 29 is computed by multiplying each term of column 22 by the corresponding value of z in column 19. F is equal to one-half the sum of column 29 and its value should be set down in the space provided below Table 4. Values of other expressions below Table 4 are found as indicated and require no particular explanation.

Table 5. Computations for V_o .— V_o is computed from the formula

$$V_o = \frac{\frac{1}{2} \sum_a^l (z-k)\Delta \left(z - \frac{\sum z\Delta}{\sum \Delta} \right) - \frac{G}{C} \frac{1}{2} \sum_a^l (z-k)\Delta \left(y - \frac{\sum y\Delta}{\sum \Delta} \right)}{F - \frac{BG}{C}}$$

Column 31 is copied from column 24. Column 32 is computed by multiplying each term in column 17 by $\frac{G}{C}$. Column 33 is computed by subtracting each term in column 32 from the corresponding term in column 31 and the numerical work may be checked by subtracting the sum of column 32 from the sum of column 31. Column 34 is computed by dividing each term in column 33 by $\left(F - \frac{BG}{C} \right)$. Column 34 now contains the values of V_o for a load of unity placed successively at each load point. The top figure in column 34, that is the value of V_o for a unit load at point 1, should equal unity, but allowance should be made for the inaccuracy due to dropping of decimals.

TABLE 5. *Computations for V_o* TABLE 6.—*Computations for H_o* TABLE 7.—*Computations for M_o*

Points	31	32	33	34	35	36	37	38	39	40	41	42	43	44
	$\frac{1}{2} \sum_a (z-k)\Delta$ $(z-\Sigma z\Delta)$	G Col. 31— Col. 32	Col. 31— Col. 32	$V_o =$ Col. 33 $F - \frac{BG}{C}$	$V_o B$ Col. 35— Col. 17		$H_o =$ Col. 36 C	Δ	$\frac{1}{2} \sum_a$	$\frac{1}{2} \sum_a$ $(z-k)\Delta$	$\frac{dx}{\Sigma \Delta} \times$ Col. 40	$\frac{H_o}{\Sigma \Delta} \times$ $\frac{dx \Sigma z \Delta}{2 \Sigma \Delta}$	$V_o \frac{dx \Sigma z \Delta}{2 \Sigma \Delta}$	$M_o =$ Col. 41 + Col. 42— Col. 43
0	19,309.05	+2,954.34	17,252.71	1.0000	2,347.96	0.00	0.000	5.23	477.93	4,736.59	34.312	0.000	36.062	-1.750
1	19,296.51	+1,987.86	17,218.65	.9980	2,343.27	73.50	.018	7.99	439.94	4,258.66	30.850	.849	35.990	-4.200
2	18,963.29	+1,845.60	17,116.69	.9921	2,329.42	229.94	.144	12.02	457.92	3,788.72	27.445	2.520	35.777	-5.812
3	18,532.48	+1,632.16	16,900.32	.9796	2,300.07	436.44	.385	17.90	440.02	3,330.80	24.128	4.988	35.326	-6.210
4	17,858.10	+1,348.11	16,509.99	.9569	2,246.77	707.48	.462	23.67	416.35	2,890.78	20.941	8.085	34.507	-5.481
5	16,999.00	+1,013.45	15,895.55	.9213	2,163.18	1,006.00	.657	29.20	387.05	2,474.43	17.925	11.498	33.224	-3.801
6	15,678.42	+655.17	15,023.25	.8708	2,044.61	1,296.52	.846	30.17	356.88	2,087.38	15.121	14.806	31.403	-1.476
7	14,218.34	+301.62	13,916.72	.8066	1,893.87	1,549.47	1.011	31.58	325.70	1,730.70	12.536	17.693	29.047	+1.142
8	12,581.21	-21.53	12,602.74	.7205	1,715.19	1,739.77	1.136	32.87	292.43	1,405.20	10.179	19.881	26.343	+3.717
9	10,825.52	-294.03	11,119.55	.6445	1,513.26	1,848.99	1.207	34.31	258.12	1,112.77	8.061	21.124	23.242	+5.943
10	9,011.70	-499.85	9,511.55	.5515	1,294.90	1,865.64	1.218	36.46	221.66	854.65	6.191	21.316	19.888	+7.619
11	7,218.23	-628.44	7,846.67	.4548	1,067.85	1,785.41	1.165	36.46	185.20	632.99	4.585	20.388	16.401	+8.572
12	5,709.01	-677.21	6,386.22	.3586	841.98	1,615.23	1.054	36.56	148.64	447.79	3.244	18.446	12.932	+8.758
13	3,930.41	-647.56	4,607.97	.2671	627.14	1,366.54	.892	34.71	113.93	299.15	2.167	15.611	9.632	+8.146
14	2,633.72	-553.50	3,187.22	.1847	433.67	1,065.67	.696	31.58	82.35	185.22	1.342	12.181	6.661	+6.862
15	1,572.10	-417.98	1,990.08	.1153	270.72	747.98	.488	28.91	53.44	102.87	.745	8.540	4.158	+5.127
16	810.94	-268.55	1,079.49	.0626	146.98	453.62	.296	23.37	30.07	49.43	.358	5.180	2.257	+3.281
17	339.41	-137.56	476.97	.0276	64.80	221.87	.145	16.22	13.85	19.36	.140	2.538	.995	+1.683
18	101.35	-47.77	149.12	.0086	20.19	74.74	.049	8.34	5.71	5.51	.040	.857	.310	+0.587
20	0.00	0.00	0.00	.00	0.00	0.00	.00	5.51	0.00	0.00	0.000	0.000	0.000	0.000
0'	195,241.79	6,647.33	188,594.46	10.9311	25,665.83	18,075.81	11.799				30,412.80	220.310	206.192	394.195 + 32.607

$$V_o = \frac{1}{2} \sum_a (z-k)\Delta \left(z - \frac{\Sigma z \Delta}{\Sigma \Delta} \right) - \frac{G}{C} \frac{1}{2} \sum_a (z-k)\Delta \left(y - \frac{\Sigma y \Delta}{\Sigma \Delta} \right)$$

$$F - \frac{BG}{C}$$

$$H_o = \frac{V_o b - \frac{1}{2} \sum_a (z-k)\Delta \left(y - \frac{\Sigma y \Delta}{\Sigma \Delta} \right)}{C}$$

Table 6. *Computations for H_o .*— H_o is computed from the formula,

$$H_o = \frac{V_o b - \frac{1}{2} \sum_a (z-k)\Delta \left(y - \frac{\Sigma y \Delta}{\Sigma \Delta} \right)}{C}$$

$V_o b$ is computed by multiplying each term in column 34 by the value of B and entered in column 35. The term $\frac{1}{2} \sum_a (z-k)\Delta \left(y - \frac{\Sigma y \Delta}{\Sigma \Delta} \right)$ has already been recorded in column 17, so the numerator of the equation for H_o is computed by subtracting each term of column 17 from the corresponding term of column 35. The numerical work is checked by subtracting the sum of column 17 from the sum of column 35. H_o is then computed by dividing each term in column 36 by C . The top figure of column 35 should be equal to the top figure of column 17 and the top figure of column 36, should be zero.

Table 7. *Computations for M_o .*— M_o is computed from the formula,

$$M_o = \frac{dx}{\Sigma \Delta} \frac{1}{2} \sum_a (z-k)\Delta + H_o \frac{\Sigma y \Delta}{\Sigma \Delta} - V_o \frac{dx \Sigma z \Delta}{2 \Sigma \Delta}$$

For convenience the values of Δ are copied in column 38 from column 11. Columns 39 and 40 are computed by summing up column 38 in the same way as previously explained for other similar columns. Columns 41, 42 and 43 are computed as indicated by the headings and no further explanation is necessary. M_o is computed in column 44 by adding the corresponding members of columns 41 and 42 and subtracting the

corresponding member of column 43. M_o for a unit load at point 1 is always numerically equal to $\frac{dx}{2}$ and the numerical work in computing column 44 may be checked from the totals of columns 41, 42 and 43.

Tables 8, 9, and 10. *Computations for moments at points 2, 11 and 0'.*—Having determined the values of H_o , V_o , and M_o , other points must now be selected for detailed examination as to stress conditions. In this case points 2 and 11 have been selected and point 0' should always be included. Values of H_o , V_o and M_o are copied in columns 46, 47 and 48.

The bending moment at any point is computed from the formula

$$M_x = M_o + m_x - H_o y.$$

For any particular point, z and y are constant and k depends on the position of the load. For point 2, z is equal to 3 and the value of y is taken from column 12 of Table 2.

Column 49 is computed by multiplying the values of V_o in column 47 by z . This is done only for the points to the left of the one for which the moment is being computed because the formula $m_x = [V_o z - (z-k)] \frac{dx}{2}$ is used only where z is greater than k . For the points to the right, m_x is found by the formula $m_x = V_o z \frac{dx}{2}$. Column 50 is computed by subtracting $(z-k)$ from the values of $V_o z$ in column 49. For a unit load at point 1, $V_o z - (z-k)$ will be unity in all cases so its value is permanently printed in the tables. In column 51, M_2 is computed for points 1 and 2 by multiplying

TABLE 8.—Computations for M_2 TABLE 9.—Computations for M_{11} TABLE 10.—Computations for M_o'

	45	46	47	48	49	50	51	52	53	54	55	56	57	58	59	60	61	62	63								
Points	Point 2, z=3; y=7.10																			Point 11, z=21; y=19.92			Point 0', z=40; y=5.00				
	H_o	V_o	M_o	$V_o z$	$V_o z - (z-k)$	m_2	$H_o y$	M_2	$V_o z$	$V_o z - (z-k)$	m_{11}	$H_o y$	M_{11}	$10 V_o$	$10 V_o - (40-k)$	$m_{o'}$	$H_o y$	$M_{o'}$									
0	0.000	1.0000	-1.750	3.0000	1.0000	1.750	0.000	0.000	21.0000	1.0000	1.750	0.000	0.000	40.0000	+1.0000	+1.750	0.000	0.000									
1	.018	.9989	-1.300	2.9940	2.9910	5.240	.341	+ .599	20.9580	2.9580	5.176	.956	-.080	39.9200	+2.9200	+5.110	.240	+ .570									
2	.144	.9921	-5.812			5.209	1.022	-1.625	20.8341	4.8341	8.460	2.868	-.220	39.6840	+4.6840	+8.197	.720	+1.665									
3	.285	.9796	-6.210			5.143	2.024	-3.091	20.5716	6.5716	11.500	5.677	-.387	39.1840	+6.1840	+10.822	1.425	+3.187									
4	.432	.9569	-5.481			5.024	3.280	-3.737	20.0949	8.0949	14.156	9.203	-.518	38.2760	+7.2760	+12.733	2.310	+4.942									
5	.657	.9213	-3.801			4.837	4.665	-3.629	19.3473	9.3473	16.358	13.087	-.530	36.8520	+7.8520	+13.741	3.285	+6.655									
6	.843	.8708	-1.476			4.572	6.007	-2.911	18.2868	10.2868	18.002	16.852	-.326	34.8320	+7.8320	+13.706	4.230	+8.000									
7	1.011	.8036	+1.142			4.235	7.178	-1.801	16.9386	10.9386	19.143	20.139	+.146	32.2640	+7.2640	+12.712	5.055	+8.799									
8	1.136	.7305	+3.717			3.835	8.066	-.514	15.3405	11.3405	19.846	22.629	+.934	29.2200	+6.2200	+10.885	5.680	+8.922									
9	1.207	.6445	+5.943			3.384	8.570	+.757	13.5345	11.5345	20.185	24.043	+2.085	25.7800	+4.7800	+8.365	6.035	+8.273									
10	1.218	.5515	+7.619			2.895	8.648	+1.866			20.268	24.263	+3.624	22.0600	+3.0600	+7.355	6.090	+6.884									
11	1.165	.4548	+8.572			2.388	8.271	+2.689			16.714	23.207	+2.079	18.1920	+1.1920	+2.086	5.825	+4.833									
12	1.054	.3586	+8.758			1.883	7.483	+3.158			13.179	20.996	+.941	14.3440	-.6560	-1.148	5.270	+2.340									
13	.892	.2671	+8.146			1.402	6.333	+3.215			9.816	17.769	+.193	10.6840	-2.3160	-4.053	4.460	-.367									
14	.696	.1847	+6.862			.970	4.942	+2.890			6.788	13.864	-.214	7.3880	-3.6120	-6.321	3.480	-2.939									
15	.488	.1153	+5.127			.605	3.465	+2.267			4.237	9.721	-.357	4.6120	-4.3880	-7.679	2.440	-4.992									
16	.293	.0626	+3.281			.329	2.102	+1.508			2.301	5.896	-.314	2.5040	-4.4960	-7.868	1.480	-6.067									
17	.145	.0276	+1.683			.145	1.030	+.798			1.014	2.888	-.191	1.1010	-3.8960	-6.818	.725	-5.860									
18	.049	.0086	+.587			.045	.348	+.284			.316	.976	-.073	.3440	-2.6560	-4.648	.245	-4.306									
19	0.000	0.0000	0.000	0.000	0.0000	0.000	0.000	0.000	0.000	0.0000	0.000	0.000	0.000	0.0000	-1.0000	-1.750	0.000	-1.750									
0'	11.799	10.9311	+32.607			3.891	83.775	+2.723			209.219	235.034	+6.792			+65.177	58.995	+38.789									

$$M = M_o + m_x - \frac{H_o y}{z > k} \frac{dx}{2}$$

$$m_x = \left[\frac{V_o z - (z-k)}{2} \right]$$

For check:

$$\begin{aligned} \Sigma \text{Col. 48} + \Sigma \text{Col. 51} - \Sigma \text{Col. 52} &= \Sigma \text{Col. 53.} \\ \Sigma \text{Col. 48} + \Sigma \text{Col. 56} - \Sigma \text{Col. 57} &= \Sigma \text{Col. 58.} \\ \Sigma \text{Col. 48} + \Sigma \text{Col. 61} - \Sigma \text{Col. 62} &= \Sigma \text{Col. 63.} \end{aligned}$$

the figure in column 50 by $\frac{dx}{2}$. For the other points m_2 is computed by multiplying the value of V_o in column 47 by $\frac{dx}{2}$.

$H_o y$ is computed by multiplying the values of H_o in column 46 by the value of y for the point under consideration. M_2 in column 53 is computed by adding the corresponding terms in columns 48 and 51 and subtracting the corresponding term in column 52. The numerical work may be checked from the totals of those columns. M_x for a load at point 1 will be zero in all cases because, for a unit load at point 1, M_o is equal $-\frac{dx}{2}$ and m_x is equal to $+\frac{dx}{2}$ and H_o is equal to zero.

Tables 9 and 10 are compiled in exactly the same manner as Table 8. Since M_o' should always be computed the numerical value of z is used in the column headings.

Table 11. Computations for thrust, shears, and moments due to dead load.—This table is filled out in the same manner as Table 8 of part 1. The values of H_o , V_o , M_o , M_2 , M_{11} , and M_o' are copied from columns 37, 34, 44, 53, 58, and 63 in columns 65 to 70. The weight of the dead load applied at each point is computed as shown below and tabulated in column 71. The weight of the concrete is assumed to be 150 pounds per cubic foot and the fill above the arch ring 110 pounds per cubic foot. The dead load applied at each point is therefore equal to $150 h ds + 110 h_f dx$, in which h is the depth of the fill above the arch ring at each point and may be scaled from the drawing. Values of h and ds are found in columns 2 and 10 of Table 1. After column 71 is filled out the table is completed by multiplying the coefficients for a load of unity in the preceding columns by the actual dead load in order to get the values in the remaining columns of the table.

Calculation of dead load

Point	h (ds)	$150 h(ds)$	h_f	$110 h_f dx$	Dead load
1	14.05	2,105	17.1	6,600	8,700
2	10.20	1,530	13.3	5,130	6,660
3	7.45	1,115	10.1	3,900	5,015
4	5.50	825	7.7	2,960	3,785
5	4.72	708	5.9	2,270	2,978
6	4.13	620	4.5	1,730	2,350
7	3.95	592	3.1	1,310	1,902
8	3.82	573	2.5	960	1,533
9	3.70	555	1.8	690	1,245
10	3.57	535	1.3	500	1,035
11	3.50	525	1.0	385	910
12	3.50	525	1.0	385	910
13	3.51	527	1.0	385	912
14	3.61	541	1.4	540	1,081
15	3.82	573	2.0	770	1,343
16	4.07	610	2.9	1,120	1,730
17	4.66	700	4.3	1,660	2,360
18	5.95	893	6.1	2,350	3,243
19	9.00	1,350	8.5	3,270	4,620
20	13.35	2,003	12.1	4,650	6,653

Table 12. Computations for maximum stresses. This table is the same as Table 9 (pt. 1) for symmetrical arches and is filled out in the same way except for the differences in the formulas due to the lack of symmetry. Column 78 is filled out as explained for column 75 of the symmetrical arch forms. The dead load moments, thrusts, and shears are found in Table 11, and copied in the proper places in columns 80, 81, and 82.

The live load has been assumed as 125 pounds per square foot which gives $125 dx = 437.5$ pounds per load point. As in the case of the symmetrical arch we wish to find the maximum stress at each point and the stress is a function of the moment, thrust and shear and as has been pointed out it is not necessarily maximum at the same time that the moment is, but it is so nearly so that this is assumed to be the case. The maximum positive live load moment at point 0 is found by multiplying the sum of all of the positive quantities in column 67 by 437.5. The horizontal thrust which

TABLE 11.—Computations for H_o , V_o , and M due to dead load

64	65	66	67	68	69	70	71	72	73	74	75	76	77	
Points	Unit load							D. L.	Dead load					
	H_o	V_o	M_o	M_2	M_{11}	M_o'	H_o		V_o	M_o	M_2	M_{11}	M_o'	
1	0	1.0000	1.750	0	0	0	8,700	0	8,700	15,225	0	0	0	
2	.048	.9980	4.300	+.599	-.080	+.570	6,660	320	6,647	28,638	+.3,989	-.533	+.3,796	
3	.144	.9921	5.812	1.625	-.220	1.665	5,015	722	4,975	29,147	8,149	1,103	8,350	
4	.285	.9796	6.210	3.091	-.387	3.187	3,785	1,079	3,708	23,505	11,699	1,465	12,063	
5	.462	.9569	5.481	3.737	-.518	4.942	2,980	1,377	2,852	16,333	11,136	1,544	14,727	
6	.657	.9213	3.801	3.629	-.530	6.655	2,350	1,544	2,165	8,932	8,528	1,246	15,639	
7	.846	.8708	1.476	2.911	-.326	8.000	1,900	1,607	1,655	2,804	5,531	619	15,200	
8	1.011	.8066	1.142	1.801	-.146	8.799	1,530	1,547	1,234	1,747	2,756	223	13,462	
9	1.136	.7305	3.717	.514	-.934	8.922	1,245	1,414	909	4,628	640	1,163	11,108	
10	1.207	.6445	5.943	.757	2.085	8.273	1,035	1,249	667	6,151	783	2,158	8,563	
11	1.218	.5515	7.619	1.866	3.624	6.884	910	1,108	502	6,933	1,698	3,298	6,264	
12	1.165	.4548	8.572	2.689	2.079	4.833	910	1,060	414	7,800	2,447	1,892	4,398	
13	1.054	.3586	8.758	3.158	.941	2.340	915	964	328	8,014	2,890	861	2,141	
14	.892	.2671	8.146	3.215	-.193	-.367	1,080	963	288	8,798	3,472	208	396	
15	.696	.1847	6.862	2.890	-.214	2.939	1,345	936	248	9,229	3,887	288	3,953	
16	.488	.1153	5.127	2.267	-.357	4.992	1,730	844	199	8,870	3,922	618	8,636	
17	.296	.0626	3.281	1.508	-.314	6.067	2,360	699	148	7,743	3,559	741	14,318	
18	.145	.0276	1.683	.798	-.191	5.860	3,245	471	90	5,461	2,590	620	19,016	
19	.049	.0086	.587	.284	-.073	4.306	4,620	226	40	2,712	1,312	337	19,894	
20	0	0	0	0	0	1.750	6,650	0	0	0	0	0	11,638	
	11.799	10.9311	61.437	20.031	10.002	65.070	58,965	18,130	35,769	78,086	30,549	9,803	115,711	
			28.839	17.308	3.210	26.281				124,584	48,439	9,114	77,851	
										46,498	17,890	689	37,860	

TABLE 12.—Computations for unit stresses

78	79	80	81	82	83	84	85	86	87	88	89	90	91	92	93			
										Extrados	Intrados	Maximum stresses						
										$N + M \frac{h}{2I}$	$N - M \frac{h}{2I}$	Extrados		Intrados				
												+	-	+	-			
Point 0	D. L.	18,130	46,498	35,769	9,953	29,903	39,856	106	280	174	386	174	174	386	386			
sin $\phi=0.836$.	+C. L. L.	4,094	26,879	1,843	2,248	1,541	3,789	10	162	172	152	172			152			
cos $\phi=0.549$.	-C. L. L.	1,068	12,613	2,940	586	2,458	3,044	8	76	68	84	68	68	84				
Area=377.	-U. L. L.	761	11,335	55	418	46	464	1	68	69	67	69			67			
$\frac{h}{2I}=0.00603$.	+T.	1,014	15,113	73	557	61	618	2	91	93	89		93	89				
										Total excluding temperature stresses		Total		+	67	242	559	167
Point 2	D. L.	18,130	17,890	27,069	12,075	20,193	32,268	110	180	70	290	70	70	290	290			
sin $\phi=0.746$.	+C. L. L.	3,175	8,764	1,607	2,115	1,199	3,314	11	88	99	77	99			77			
cos $\phi=0.666$.	-C. L. L.	1,987	7,572	2,738	1,323	2,043	3,366	12	76	64	88	64	64	88				
Area=293.	-U. L. L.	761	6,221	55	507	41	548	2	62	64	60	64	64	60	60			
$\frac{h}{2I}=0.01005$.	+T.	1,014	8,297	73	675	54	729	3	83	86	80		86	80	80			
										Total excluding temperature stresses		Total		+	93	134	458	153
Point 11	D. L.	18,130	689	569	18,112	26	18,138	113	25	138	88	138	138	88	88			
sin $\phi=0.046$.	+C. L. L.	3,361	4,376	356	3,358	16	3,374	21	158	179	137	179			137			
cos $\phi=0.999$.	-C. L. L.	1,801	1,404	51	1,799	2	1,801	11	51	40	62		40	62				
Area=161.	-U. L. L.	761	1,803	55	760	3	763	5	65	60	70		60	70				
$\frac{h}{2I}=0.0361$.	+T.	1,014	2,403	73	1,013	3	1,016	6	87	81	93	81			93			
										Total excluding temperature stresses		Total		+	398	38	220	49
Point 0'	D. L.	18,130	37,860	23,196	10,117	19,253	29,370	78	228	306	150	306	306	150	150			
sin $\phi=0.830$.	+C. L. L.	4,039	28,468	1,197	2,254	994	3,248	9	172	181	163	181			163			
cos $\phi=0.558$.	-C. L. L.	1,123	11,498	2,771	627	2,300	2,927	8	69	61	77		61	77				
Area=377.	-U. L. L.	761	11,380	55	425	46	379	1	69	70	68	70			68			
$\frac{h}{2I}=0.00603$.	+T.	1,014	15,153	73	566	61	505	1	91	92	90		92	90				
										Total excluding temperature stresses		Total		+	557	153	17	313

$$N = H \cos \phi + V \sin \phi$$

$$f_c = \frac{N}{A} \pm M \frac{h}{2I}$$

occurs at the same time as this maximum moment is found by adding up the quantities in column 65 for the points which give a positive moment and multiplying the sum by 437.5. The vertical shear which occurs at the same time as the maximum moment is found in the same way. The maximum negative moment and the thrust and shear which occur at the same time are found by placing loads at all of the points which give negative moments. These values are recorded in the proper places in Table 12.

The live-load moments and thrusts at the other points are found in the same way. In determining the shear for the other points it should be remembered that V_o in column 66 is the coefficient for the left reaction which is equal to the coefficient for the vertical shear at point 0, but not at other points if there are loads between the point where the shear is desired and the left reaction. If there are any loads to the left of the point under consideration, the shear is found by adding up the proper quantities in column 66 and then subtracting 1.00 for each load to the left. The quantity thus obtained is multiplied by the live load per load point, as shown in the following computations:

Live-load moments, thrusts, and shears

Live load = 125 pounds per square foot.
= $125 \times 3.5 = 437.5$ pounds per load point

Point 0:	
+ $M = 437.5 \times 61.437 =$	26,879
$H = 437.5 \times 9.357 =$	4,094
$V = 437.5 \times 4.212 =$	1,843
- $M = 437.5 \times 28.830 =$	12,613
$H = 437.5 \times 2.442 =$	1,068
$V = 437.5 \times 6.719 =$	2,940
Point 2:	
+ $M = 437.5 \times 20.031 =$	8,764
$H = 437.5 \times 7.258 =$	3,175
$V = 437.5 \times (4.673 - 1.00) =$	1,607
- $M = 437.5 \times 17.308 =$	7,572
$H = 437.5 \times 4.541 =$	1,987
$V = 437.5 \times 6.258 =$	2,738
Point 11:	
+ $M = 437.5 \times 10.062 =$	4,376
$H = 437.5 \times 7.683 =$	3,361
$V = 437.5 \times (3.813 - 3.0) =$	356
- $M = 437.5 \times 3.210 =$	1,404
$H = 437.5 \times 4.116 =$	1,801
$V = 437.5 \times (7.117 - 7.0) =$	51
Point 0':	
+ $M = 437.5 \times 65.070 =$	28,468
$H = 437.5 \times 9.233 =$	4,039
$V = 437.5 \times (10.265 - 13.0) =$	1,197
- $M = 437.5 \times 26.281 =$	11,498
$H = 437.5 \times 2.566 =$	1,123
$V = 437.5 \times (0.666 - 7.0) =$	2,771

These moments, thrusts, and shears are tabulated in the proper places in columns 80, 81, and 82.

The values of V_t are found from the equation

$$V_t = \frac{2r}{(dx)^2} + 20 \frac{G}{C} etE$$

$$F - B \frac{G}{C}$$

In substituting in this equation, e , the coefficient of expansion of the concrete, is assumed to be 0.000006. E , the modulus of elasticity of the concrete, is taken as 2,000,000 pounds per square inch or 288,000,000 pounds per square foot, and t has been assumed as $+30^\circ$ F. and -40° F. These values with other values already determined when substituted in the equation give values of V_t as $+55$ and -73 .

The formula for H_t given on page 95 reduces to

$$H_t = \frac{V_t B + 34,560t}{C} = -1,014$$

The values of V_t and H_t are the same for every point.

M_t is different for every point and substitution in the formula on page 95 gives the following:

For point 0,

$$M_{t,0} = -V_t \frac{dx}{2} \Sigma \Delta + H_t \Sigma \Delta = +11,335$$

$$-15,113$$

For point 2,

$$M_{t,2} = M_{t,0} + 3V_t \frac{dx}{2} - 7.1H_t = +6,221$$

$$-8,297$$

For point 11,

$$M_{t,11} = M_{t,0} + 21V_t \frac{dx}{2} - 19.92H_t = -1,803$$

$$+2,403$$

For point 0',

$$M_{t,0'} = M_{t,0} + 40V_t \frac{dx}{2} - 5H_t = +11,380$$

$$-15,153$$

Table 12 is completed in the same manner as the similar table for the symmetrical arch.

Stresses in steel and concrete.—As in the example for the symmetrical arch, the maximum stresses in Table 12 should be examined to determine if the tensile strength of the concrete will be exceeded. In the present example the tension in the concrete at the extrados at point 0 is 335 pounds per square inch, the concrete will crack and the tension must be taken by the steel. The stresses should therefore be computed in accordance with the theory of flexure and direct stress. The calculations for point 0 differ from those given in the example for a symmetrical arch as there is reinforcement in both the top and bottom of the arch ring. In this case the diagrams on page 399 to 402 of Hool and Johnson's Concrete Engineers' Handbook are used and it will be assumed that the reader is familiar with these diagrams and the theory related to them. It should be noted that x_o and t in Hool and Johnson's diagrams are the same as u and h in this article.

The following calculations are made to determine the stress in steel and concrete at point 0. The worst condition at point 0 is caused by a negative moment due to dead load, maximum negative moment due to live load, and a negative moment due to a fall of temperature.

	M_o	N
D. L.	46,498	+39,856
- L. L.	-12,613	+3,044
- T.	-15,113	-618
	-74,224	+42,282

$$u = \frac{-74224}{+42282} = -1.76 \text{ feet, eccentricity of thrust.}$$

$$\frac{u}{h} = \frac{1.76}{2.5} = 0.70 \text{ feet (value of } \frac{x_o}{t} \text{ in Hool and Johnson).}$$

$$\frac{d'}{h} = \frac{.17}{2.5} = 0.068 \left\{ \begin{array}{l} (d' = \text{distance from center of steel} \\ \text{to surface of concrete).} \end{array} \right.$$

$$P_o = \frac{2 \times .5625}{144 \times 2.5} = .0031 \text{ per cent steel.}$$

From the diagram on page 399 of Hool and Johnson (for $d' = 0.05t$ in which t is the same as h) with $P_o = 0.0031$ and $\frac{u}{h} = 0.70$, k is equal to 0.310, and from diagram on page 400 (for $d' = 0.10t$), k is equal to 0.300. As $\frac{d'}{h}$ is 0.068 we should use a value of k between the two or about 0.305.

(Continued on page 104)

COMPARISON OF TRUCK AND RAILROAD TONNAGE BETWEEN COLUMBUS AND SELECTED OHIO CITIES

AN EXTRACT FROM THE REPORT ON A SURVEY OF TRANSPORTATION ON THE STATE HIGHWAY SYSTEM OF OHIO

IN order to determine what proportion of the total tonnage of commodities moving between cities is hauled by motor truck, and also to develop the factors influencing the choice of motor truck or railroad, an analysis was made of the net tonnage hauled between Columbus and 34 Ohio cities by motor truck and rail lines.

The cities selected are located from 7 to 134 highway miles from Columbus and were chosen to permit an analysis of the effect of length of haul upon the proportions of tonnage transferred by motor truck and railroad, respectively. Hillsboro and Johnstown, both having indirect rail connections with Columbus, were selected to determine the effect of indirect rail connections upon motor-truck transportation. Commercial Point, Dublin, Reynoldsburg, and Rome were selected to ascertain the amount of tonnage hauled by truck between points having no railroad facilities.

Motor truck net tonnage and railroad carload and less-than-carload tonnage data for an average month of 1925 were taken as the basis for this comparison, as shown in Figure 1, the size of the circles for each city indicating the total net tonnage transported.

It is apparent that distance is an important factor in the amount of tonnage hauled by motor truck. Between Columbus and Akron, Cincinnati, and Toledo, distances of over 100 miles, a very small part of the total tonnage is hauled by truck, while between Columbus and Grove City and between Columbus and Alton, distances of 8 and 9 miles, respectively, almost all of the tonnage is transported by motor truck.

Table 1 presents a summary of the relation between motor truck and rail tonnage according to length of haul.

TABLE 1.—Proportion of motor truck and railroad net tonnage according to length of haul for average month, 1925¹

Length of haul (highway miles)	Motor truck		Rail (carload)		Rail (less than carload)		Total	
	Tons	Per cent	Tons	Per cent	Tons	Per cent	Tons	Per cent
Less than 20	6,091	84.5	1,112	15.4	10	0.1	7,213	100.0
20-39	5,973	54.7	4,803	44.0	145	1.3	10,921	100.0
40-59	2,299	32.0	4,484	62.4	404	5.6	7,187	100.0
60-99	980	24.2	2,409	59.4	663	16.4	4,052	100.0
100 and over	157	2.3	5,280	77.4	1,383	20.3	6,820	100.0

¹ Based upon tonnage between Columbus and 30 cities having rail connections.

Although other factors besides length of haul influence the proportions of total tonnage hauled by motor truck and rail lines, respectively, and although the number of cities in each zone of haul (Table 1) is not large, there is clearly indicated the tendency for the proportion of motor-truck tonnage to decrease with increase in distance. For hauls of less than 20 miles, 84.5 per cent of the total tonnage is transported by motor truck; between 20 and 39 miles, motor-truck tonnage is 54.7 per cent of total tonnage; and for the longer

hauls, motor-truck tonnage is 32, 24.2, and 2.3 per cent, respectively. As the percentage of motor-truck tonnage decreases with increase in distance both rail, carload, and less-than-carload tonnage increase. No appreciable amount of less-than-carload tonnage is noted under 40 miles. Between 40 and 59 miles the less-than-carload tonnage is 5.6 per cent of the total and this percentage increases to 20.3 for distances of 100 miles or more.

Among other factors controlling the proportion of total tonnage hauled by truck are the type of commodities and rail facilities. An illustration of the former is indicated in the movement between Columbus and Johnstown. Although Johnstown is only 22 miles from Columbus by highway and 41 miles by railroad, 73.1 per cent of the tonnage is hauled by rail, practically all of which moves in carload lots. This high percentage of rail tonnage, considering the comparatively short highway mileage and the longer rail connection, is due to the type of commodity transported, 95.7 per cent of the total tonnage being gravel, sand, and stone. These commodities can not be economically hauled by motor truck for this distance, as is indicated by the fact that the average length of haul for trucks hauling gravel, sand, and stone in Ohio is only 10 miles.

The influence of indirect rail connection upon the proportion of motor truck and rail tonnage is shown between Columbus and Hillsboro, a distance of 65 miles by highway and 97 miles by railroad. Between these points 47.5 per cent of the total tonnage is carried by motor truck. Reference to Table 1 shows that, under normal conditions, between 40 and 59 miles, only 32 per cent of the total tonnage is carried by motor truck, and that between 60 and 99 miles the corresponding figure is 24.2 per cent. It is evident that the relatively high percentage of motor-truck tonnage between Columbus and Hillsboro is due, in large part at least, to the fact that the distance by highway is 32 miles shorter than the distance by railroad.

EFFECT OF SALTS IN MIXING WATER ON STRENGTH OF MORTAR STUDIED

The bureau of engineering research of the University of Texas is now studying the results of a series of tests to determine the effect of various salts in mixing water on the strength of cement mortar. These tests have been made over a period of three years and a report is to be made soon.

The bureau has recently moved into new and more commodious quarters and an enlarged program of research work is being planned for the future, according to F. E. Giesecke, director of the bureau.

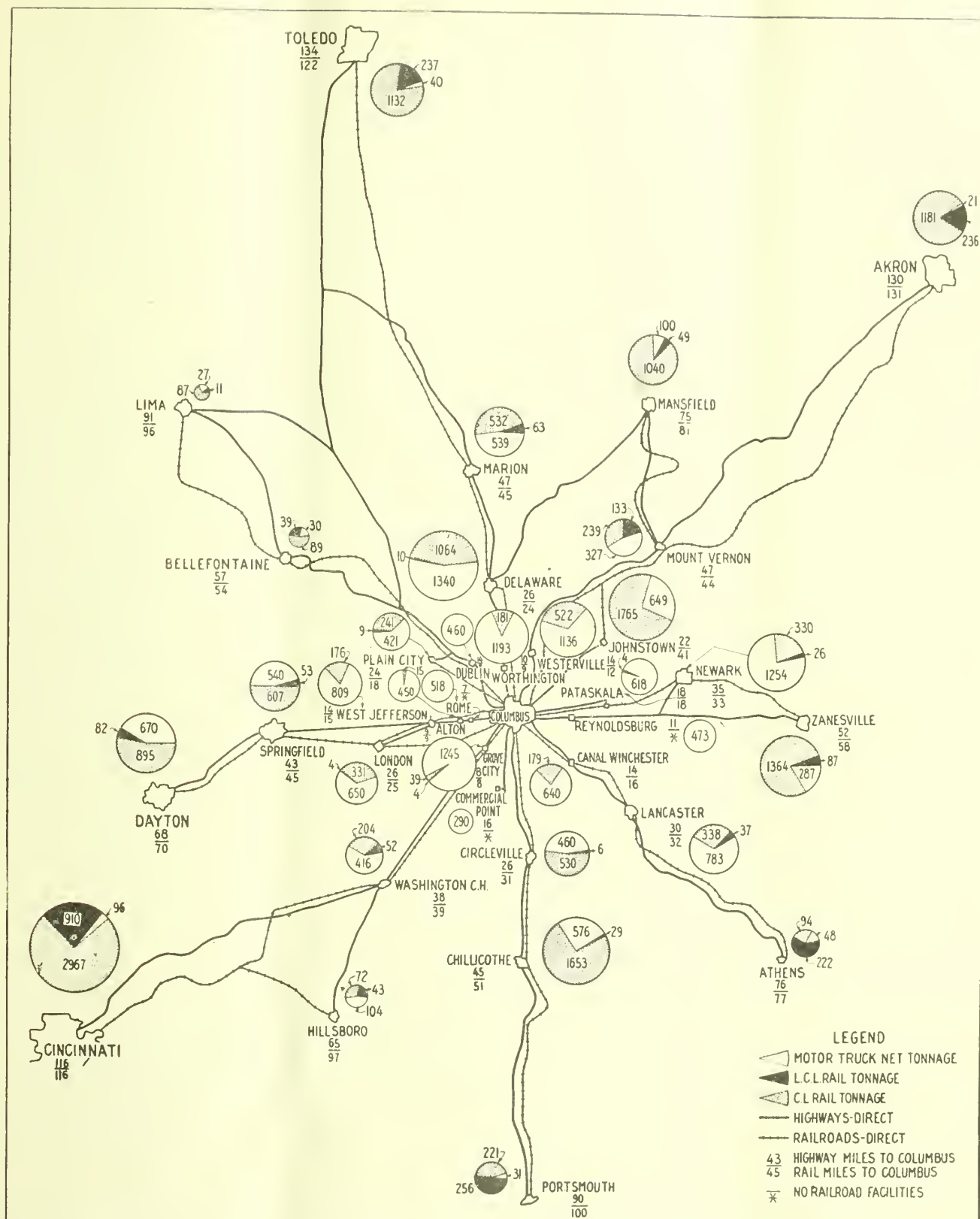


FIG. 1.—COMPARISON OF MOTOR TRUCK AND RAILROAD NET TONNAGE BETWEEN COLUMBUS AND SELECTED OHIO CITIES

YADKIN RIVER BRIDGE TEST IN PROGRESS

Preliminary studies of the effect of temperature variations on the arch ring of the bridge over the Yadkin River between Albemarle and Mount Gilead, N. C., are now in progress. The measurements recently begun constitute the first of a series of tests which will be made on the bridge, a Federal-aid structure, during the coming summer and autumn. The bridge, which will be submerged early next year by back water from a dam now under construction, is being tested under the joint auspices of the bureau and the North Carolina State Highway Commission with the aid of an advisory committee of distinguished engineers.

The temperature of the concrete is being determined at various depths in the arch ring by means of thermometers inserted in holes drilled in the arch and filled with cup grease to exclude the air. Simultaneously with the temperature readings, measurements of the deflection of the crown of the arch are made from the datum afforded by a taut piano wire. The wire is fixed at one end and passes over a pulley at the other and is kept at a uniform tension by means of a weight. The preliminary temperature studies are being made for the purpose of correcting for temperature the results obtained in subsequent loading tests.

Representatives of the advisory committee inspected the bridge on May 15 and 16 and agreed upon a program of test procedure. The loading is to consist of tanks filled with water and so constructed as to be moved readily to desired positions. Distribution of stresses will be determined for various positions of the loads. If time and sufficient funds are available, impact tests will be made, using heavy trucks for loads. Following this, an attempt will be made to load the bridge to destruction with water tanks.

The loading measurements will include the deformations of the concrete, the deflections and change in curvature of the arch ribs, and the movements of the piers. Preparations for the loading measurements are being made as rapidly as possible. A ferry will be operated across the river in order to permit closing the bridge prior to the completion of the replacing structure.

Exact measurements of the existing bridge are now being taken and an analysis of the structure will be made by the Beggs deformeter method, using a model constructed in accordance with these measurements, in order to determine the relation between the actual behavior of the structure under load and the behavior as determined by the analysis.

PRODUCTION COSTS OF BROKEN STONE

The United States Department of Agriculture has recently issued Miscellaneous Circular No. 93, Direct Production Costs of Broken Stone, by George E. Ladd, economic geologist, division of tests, Bureau of Public Roads. This circular contains a detailed report of cost analyses of operations at a number of quarries of various sizes, involving production of broken stone in various kinds of rock, and discusses generally the conditions which affect costs. The report describes in detail the character of each of the quarries studied and gives direct costs of the various operations, such as stripping, drilling face, breaking boulders by various methods,

face blasting, etc. Some quarries, where methods of operation were changed after the studies were made, were studied during the following season in order to compare the two different methods of operation. Tables of detailed costs for each quarry are followed by tables and discussions analyzing each of the major operations for all of the quarries studied. The bulletin will be of great value to quarry operators, engineers, and others having to do with the production of broken stone in determining if operations are being efficiently conducted by comparison with other quarries operating under similar conditions. It will also be useful in determining the efficiency of operations in rock excavation for highway and other purposes.

It is believed that its greatest usefulness lies in the suggestions it contains for producers who wish to adopt a cost-keeping system based on units of operation.

HIGHWAY BRIDGE LOCATION

Important principles governing the location of highway bridges are discussed in United States Department of Agriculture Bulletin 1486D, Highway Bridge Location, by C. B. McCullough, bridge engineer, Oregon State Highway Commission. In this bulletin Mr. McCullough presents the results of a long experience in the location of highway bridges, discussing the subject with reference to first cost, maintenance, operation, and relation to the highway of which it is a part.

Location of minor structures is discussed in some detail, giving desirable and undesirable features in alignment of highway and culvert location. Cost considerations involved in the location of large structures are treated under the headings length of crossing, angle of crossing, foundation conditions, and permanency of channel. It is pointed out that a location at first thought to be most desirable, may, after careful investigation, prove to be more costly than some other location which is apparently less advantageous. The bulletin also discusses maintenance considerations, alignment, grade-line treatments, traffic influence on bridge location, and location over navigable waters.

(Continued from page 101)

The diagram on page 402 is for $\frac{d'}{h} = 0.10$.

To use the diagram for $\frac{d'}{h} = 0.05$, P_o should be divided by 0.790 according to instructions below the diagram.

As the value of $\frac{d'}{h}$ in this case is 0.068 we should divide P_o by about 0.84, which gives a value of $P_o = 0.0037$ for use with the diagram. Using a value of $P_o = 0.0037$ and $k = 0.305$ we find from the diagram that $L = 0.0900$. Then substituting in the equation given

by Hool and Johnson, $f_c = \frac{M}{Lbt^2} = \frac{12 \times 74,224}{0.0900 \times 12 \times (30)^2} = 916$ pounds per square inch, and $f_s = nf_c \left(\frac{d}{kt} - 1 \right) = 15 \times 916 \left(\frac{28}{305 \times 30} - 1 \right) = 28,300$ pounds per square inch.

Stresses are computed at all points where Table 12 indicates that the concrete will crack and if necessary the design of the arch ring is revised. Usually a slight modification is sufficient to keep within the limits of the specifications.

ROAD PUBLICATIONS OF BUREAU OF PUBLIC ROADS

Applicants are urgently requested to ask only for those publications in which they are particularly interested. The Department can not undertake to supply complete sets nor to send free more than one copy of any publication to any one person. The editions of some of the publications are necessarily limited, and when the Department's free supply is exhausted and no funds are available for procuring additional copies, applicants are referred to the Superintendent of Documents, Government Printing Office, this city, who has them for sale at a nominal price, under the law of January 12, 1895. Those publications in this list, the Department supply of which is exhausted, can only be secured by purchase from the Superintendent of Documents, who is not authorized to furnish publications free

ANNUAL REPORTS

- Report of the Chief of the Bureau of Public Roads, 1924.
- Report of the Chief of the Bureau of Public Roads, 1925.

DEPARTMENT BULLETINS

- No. 105D. Progress Report of Experiments in Dust Prevention and Road Preservation, 1913.
- *136D. Highway Bonds. 20c.
- 220D. Road Models.
- 257D. Progress Report of Experiments in Dust Prevention and Road Preservation, 1914.
- *314D. Methods for the Examination of Bituminous Road Materials. 10c.
- *347D. Methods for the Determination of the Physical Properties of Road-Building Rock. 10c.
- *370D. The Results of Physical Tests of Road-Building Rock. 15c.
- 386D. Public Road Mileage and Revenues in the Middle Atlantic States, 1914.
- 387D. Public Road Mileage and Revenues in the Southern States, 1914.
- 388D. Public Road Mileage and Revenues in the New England States, 1914.
- 390D. Public Road Mileage and Revenues in the United States, 1914. A Summary.
- 407D. Progress Reports of Experiments in Dust Prevention and Road Preservation, 1915.
- *463D. Earth, Sand-Clay, and Gravel Roads. 15c.
- *532D. The Expansion and Contraction of Concrete and Concrete Roads. 10c.
- *537D. The Results of Physical Tests of Road-Building Rock in 1916, Including all Compression Tests. 5c.
- *583D. Reports on Experimental Convict Road Camp, Fulton County, Ga. 25c.
- *660D. Highway Cost Keeping. 10c.
- *670D. The Results of Physical Tests of Road-Building Rock in 1916 and 1917. 5c.
- *691D. Typical Specifications for Bituminous Road Materials. 10c.
- *724D. Drainage Methods and Foundations for County Roads. 20c.
- *1077D. Portland Cement Concrete Roads. 15c.
- *1132D. The Results of Physical Tests of Road-Building Rock from 1916 to 1921, Inclusive. 10c.
- 1259D. Standard Specifications for Steel Highway Bridges adopted by the American Association of State Highway Officials and approved by the Secretary of Agriculture for use in connection with Federal-aid road work.

- 1279D. Rural Highway Mileage, Income, and Expenditures, 1921 and 1922.
- 1486D. Highway Bridge Location.

DEPARTMENT CIRCULARS

- No. 94C. TNT as a Blasting Explosive.
- 331C. Standard Specifications for Corrugated Metal Pipe Culverts.

MISCELLANEOUS CIRCULARS

- No. 62M. Standards Governing Plans, Specifications, Contract Forms, and Estimates for Federal-Aid Highway Projects.
- 93M. Direct Production Costs of Broken Stone.
- 105M. Federal Legislation Providing for Federal Aid in Highway Construction and the Construction of National Forest Roads and Trails.

FARMERS' BULLETINS

- No. *338F. Macadam Roads. 5c.
- *505F. Benefits of Improved Roads. 5c.

SEPARATE REPRINTS FROM THE YEARBOOK

- No. *739Y. Federal Aid to Highways, 1917. 5c.
- *849Y. Roads. 5c.
- 914Y. Highways and Highway Transportation.

REPRINTS FROM THE JOURNAL OF AGRICULTURAL RESEARCH

- Vol. 5, No. 17, D- 2. Effect of Controllable Variables upon the Penetration Test for Asphalts and Asphalt Cements.
- Vol. 5, No. 19, D- 3. Relation Between Properties of Hardness and Toughness of Road-Building Rock.
- Vol. 5, No. 24, D- 6. A New Penetration Needle for Use in Testing Bituminous Materials.
- Vol. 6, No. 6, D- 8. Tests of Three Large-Sized Reinforced-Concrete Slabs Under Concentrated Loading.
- Vol. 10, No. 5, D-12. Influence of Grading on the Value of Fine Aggregate Used in Portland Cement Concrete Road Construction.
- Vol. 11, No. 10, D-15. Tests of a Large-Sized Reinforced-Concrete Slab Subjected to Eccentric Concentrated Loads.

* Department supply exhausted.

UNITED STATES DEPARTMENT OF AGRICULTURE
BUREAU OF PUBLIC ROADS
STATUS OF FEDERAL AID HIGHWAY CONSTRUCTION

AS OF

JUNE 30, 1927

STATES	FISCAL YEARS 1917-1926				FISCAL YEAR 1927				BALANCE OF FEDERAL AID FUND AVAILABLE FOR NEW PROJECTS			
	PROJECTS COMPLETED PRIOR TO JULY 1, 1926				*PROJECTS UNDER CONSTRUCTION				PROJECTS APPROVED FOR CONSTRUCTION			
	TOTAL COST	FEDERAL AID	MILES		TOTAL COST	FEDERAL AID	MILES		ESTIMATED COST	FEDERAL AID ALLOTTED	MILES	
Alabama	\$ 18,226,411.34	\$ 8,725,985.09	1,298.3		\$ 1,834,960.34	\$ 889,114.85	101.9		\$ 80,614.39	\$ 40,307.19	1.9	
Arizona	10,949,874.26	5,863,772.35	729.8		1,860,072.45	583,396.92	71.0		364,876.07	347,611.63	19.1	
Arkansas	18,364,544.50	7,668,698.35	1,323.0		3,972,070.13	1,869,434.40	237.6		977,873.98	284,001.03	19.4	
California	27,142,696.90	13,003,592.30	1,058.0		7,985,072.14	3,963,434.52	248.3		7,420,526.19	3,420,728.79	151.4	
Colorado	13,905,904.64	7,127,288.18	745.0		1,591,217.27	807,010.73	84.0		6,291,181.71	3,074,539.39	276.1	
Connecticut	5,414,567.19	2,100,285.80	117.1		982,955.10	343,414.74	20.2		5,314,165.45	1,478,079.22	63.9	
Delaware	4,918,052.29	1,781,665.60	124.3		1,319,274.26	563,906.82	35.2		796,771.29	312,891.57	32.9	
Florida	3,832,680.26	1,824,352.32	132.9		1,803,550.28	1,803,550.28	112.2		8,865,112.53	3,897,634.31	136.3	
Georgia	24,791,206.97	11,664,237.86	1,794.0		7,160,239.53	3,436,994.54	379.6		8,069,503.82	3,995,165.58	333.7	
Idaho	11,061,198.14	5,882,112.70	724.7		2,154,317.31	1,133,414.46	110.8		2,648,261.40	1,650,669.48	139.1	
Illinois	44,116,611.86	20,619,995.74	1,377.7		4,422,370.30	2,161,520.86	153.1		10,166,322.21	4,867,016.84	355.8	
Indiana	29,062,375.40	11,926,302.10	2,114.6		5,243,763.46	2,469,301.65	339.6		14,294,224.35	6,230,951.47	564.8	
Iowa	32,826,601.84	12,690,489.26	1,150.6		4,615,595.97	2,140,340.29	334.6		5,739,789.74	2,635,383.13	187.0	
Kansas	20,737,706.10	8,432,082.25	758.3		2,478,594.43	1,018,612.50	116.6		9,327,382.30	3,413,033.44	42.8	
Kentucky	11,533,401.62	6,144,739.98	1,055.9		2,046,859.52	949,152.22	122.8		4,118,538.00	1,935,266.43	140.9	
Louisiana	8,747,552.76	4,192,507.38	303.6		1,817,247.30	685,945.26	64.0		2,887,243.06	927,924.74	74.8	
Maine	10,924,943.10	5,112,991.22	423.3		685,260.83	411,947.05	54.5		814,475.47	373,185.64	33.9	
Massachusetts	18,353,757.71	6,657,260.52	374.5		2,316,988.31	769,567.53	35.9		5,900,100.14	1,540,245.69	92.9	
Michigan	25,997,240.78	11,827,052.30	953.0		5,380,007.59	2,501,432.69	121.2		8,902,769.33	5,428,748.35	365.3	
Minnesota	37,170,985.95	16,586,115.56	3,181.9		7,328,682.52	3,460,029.11	461.6		7,150,569.77	2,112,898.90	370.0	
Mississippi	15,146,989.52	7,434,534.10	1,129.0		3,195,482.23	1,589,760.62	185.1		6,756,572.72	3,268,908.98	353.7	
Missouri	28,989,166.92	13,736,014.85	1,543.2		13,400,231.49	5,945,011.63	401.6		9,250,411.52	3,833,232.27	256.8	
Montana	11,400,983.81	6,333,465.89	1,064.9		1,454,011.91	953,822.80	96.7		1,115,050.00	1,435,969.48	129.8	
Nbraska	11,533,401.62	5,474,202.52	1,768.3		4,623,638.63	2,265,183.87	479.3		12,504,680.58	6,216,951.27	130.1	
Nevada	7,558,195.91	5,130,934.59	538.8		2,863,163.80	2,468,234.09	314.9		1,324,113.16	1,314,359.67	182.7	
New Hampshire	4,992,586.60	2,377,450.07	237.6		876,339.16	401,477.98	27.2		1,924,537.73	460,853.90	30.6	
New Jersey	16,346,301.01	5,098,342.21	290.3		5,891,939.07	2,397,032.27	25.0		7,415,614.44	1,700,398.55	111.3	
New Mexico	12,407,337.77	7,339,657.58	1,137.0		3,331,931.17	1,537,940.66	79.2		3,339,904.18	2,586,415.22	233.6	
New York	43,224,279.79	17,911,957.19	1,137.0		10,959,805.66	3,781,998.46	242.3		37,212,953.01	5,415,593.95	579.8	
North Carolina	27,009,419.47	11,777,337.94	1,25.9		9,296,429.74	3,341,565.23	502.7		2,896,070.16	1,296,705.92	97.6	
North Dakota	12,313,311.40	6,031,659.78	2,131.1		3,568,247.15	1,714,433.90	522.5		5,184,096.96	1,896,476.71	73.9	
Ohio	47,699,342.90	17,371,787.03	1,364.1		4,821,684.59	1,959,593.73	150.9		11,535,410.43	4,494,341.32	334.3	
Oklahoma	28,247,960.33	13,159,999.15	1,178.9		2,134,006.75	967,590.06	89.2		4,029,291.98	1,714,249.20	251.2	
Oregon	17,027,678.42	8,593,214.79	1,189.8		2,555,706.34	1,448,238.18	115.5		1,276,070.18	50,108.85	62.4	
Pennsylvania	61,366,160.80	21,560,732.04	1,938.8		16,350,023.42	4,756,688.26	345.8		15,325,234.37	4,890,636.70	323.1	
Rhode Island	15,020,635.90	1,559,929.06	96.7		1,244,797.29	439,650.00	29.3		1,291,865.67	339,055.00	22.6	
South Carolina	12,990,245.44	6,766,322.33	1,481.9		1,391,400.93	760,655.87	86.2		6,561,820.95	2,793,135.49	235.8	
South Dakota	69,183,673.48	27,440,254.72	4,920.2		5,006,192.89	4,126,705.73	546.2		1,244,074.81	670,640.89	53.1	
Tennessee	21,624,631.57	10,276,584.02	780.0		2,659,031.46	1,274,873.53	98.7		8,762,898.20	3,614,697.07	236.7	
Texas	69,183,673.48	27,440,254.72	4,920.2		5,006,192.89	4,126,705.73	546.2		1,244,074.81	670,640.89	53.1	
Utah	8,253,176.03	5,098,440.68	546.4		901,199.30	666,639.27	82.5		2,300,006.82	1,748,040.82	173.5	
Vermont	4,242,642.64	2,017,599.51	134.5		795,075.59	331,156.50	18.2		2,135,891.86	819,113.62	41.7	
Virginia	21,990,245.44	10,396,728.11	1,095.5		4,853,775.80	2,151,415.11	165.2		1,975,392.24	971,834.24	19.0	
Washington	9,078,511.63	7,782,909.46	668.6		1,105,994.34	463,642.43	42.5		237,469.59	65,000.00	10.1	
West Virginia	9,473,716.44	4,141,062.05	382.9		951,130.88	432,685.35	26.5		7,132,862.00	2,934,191.65	239.2	
Wisconsin	20,859,091.19	10,396,728.11	1,095.5		3,322,093.59	1,588,392.00	182.5		9,450,163.85	3,568,360.00	26.9	
Wyoming	10,928,302.56	6,040,697.05	1,153.5		1,098,390.99	1,098,390.99	18.5		6,961,107.62	305,736.82	19.4	
Hawaii	966,692,834.36	426,178,703.58	82,526.6		188,047,167.12	93,828,937.66	8,431.0		340,102,631.60	142,817,679.01	13,355.1	
TOTALS									56,215,409.83	22,233,674.57	2,395.6	

* Includes projects reported completed (final vouchers not yet paid) totaling: Estimated cost \$79,432,586.10 Federal aid \$4,877,220.48 Miles 3,285.1

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UNITED STATES DEPARTMENT OF AGRICULTURE
BUREAU OF PUBLIC ROADS



VOL. 8, NO. 6



AUGUST, 1927



CHEAT HAVEN DAM, PENNSYLVANIA, TREATED WITH COAL TAR

PUBLIC ROADS

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CERTIFICATE: By direction of the Secretary of Agriculture, the matter contained herein is published as administrative information and is required for the proper transaction of the public business

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PROTECTION OF CONCRETE AGAINST ALKALI

FURTHER TESTS BY BUREAU OF PUBLIC ROADS ON TREATMENT OF CONCRETE WITH TAR AND PARAFFIN

Reported by E. C. E. LORD, Petrographer, United States Bureau of Public Roads

THE investigations carried on for some years by the Bureau of Public Roads to determine the degree of protection against alkali attack afforded by treatment of concrete with tar and paraffin have been continued, and additional information has been gained regarding the physical properties of the preservatives and their behavior when applied to various types of concrete.

SCOPE OF TESTS OUTLINED

The earlier tests, results of which have already been published,¹ were carried out on Portland cement concrete cylinders (4 by 6 inches) made of Potomac River sand and gravel graded in the proportions of 1:1½:3, 1:2:4, and 1:3:6, and cured for 28 days in damp sand and about four weeks in dry air. These cylinders were treated with 6 and 10 coats of water-gas tar alone and with 10 coats of water-gas tar and a seal coat of heavy coal tar cut back with solvent naphtha to a working consistency. In later tests a lighter tar requiring no dilution has been used in place of this heavy tar.

In addition to the tar treatment samples of each mix were immersed for 24 hours in a 20 per cent paraffin-kerosene solution at about 80° F. and given four coats of this solution after the cylinders had dried out for one week.

Two batches from each mix were left untreated, one to be stored in alkali solution and the other in tar water.

These earlier samples were weighed before and after treatment and stored, four each, in porcelain-lined, covered cans containing 6,000 cubic centimeters of a 3 per cent sodium-magnesium sulphate solution.² For the first two years the solution was frequently renewed and the specimens removed at weekly, monthly, and trimonthly periods and allowed to dry out for 48 hours before returning to the alkali bath. Following the two-year period the samples were allowed to remain undisturbed in the alkali solution for the final twelve months, but with the cans partly uncovered. During this time a considerable loss by evaporation took place, resulting in an appreciable concentration of the solution and crystallization of sulphate salts in the surfaces of the cylinders. (See fig. 1.)

At the end of each year all samples were removed from the solution, weighed, and allowed to dry out for 10 days in the laboratory, when approximately constant weight was obtained. The difference between this weight and that immediately after removal from the bath indicated capillary or absorbed moisture, while the original dry weight deducted from that after immersion and drying represented essentially the secondary salts formed during the test period. These salts together with the lime dissolved from the test specimens, as well as insoluble residues found in the cans, served as a measure of alkali attack.

During the early stages of the investigation it was noted that the lime dissolved from the untreated specimens gradually decreased in quantity until after a period of about three months it was entirely removed from solution. This loss in lime was accompanied by an equivalent loss of magnesia and by the formation of lime and magnesia carbonates and calcic sulphoaluminate. It may be a matter of interest to state that the latter salt was found only as a precipitate in the cans and not in the concrete itself, indicating that the failure of concrete under the present test conditions can not be attributed to this salt. The destructive agencies have been found to consist chiefly of magnesium hydrate and gypsum which are formed immediately when free lime developed in the cement on setting, is brought in contact with solutions containing magnesium sulphate. This was especially noticeable in imperfectly seasoned concrete, where colloidal magnesium hydrate and gypsum crystals forming in the body of the concrete caused sufficient volume change to rupture the specimens within three months. In well-seasoned concrete, on the other hand, the formation of calcium carbonate during the curing period increased the density of the mass, thereby retarding the entrance of alkali solutions and rendering the concrete very resistant to alkali attack.

Upon completion of the three-year period of exposure the cylinders were photographed to show the surface condition of the concrete. Figure 1 shows the condition of the samples of 1:2:4 concrete at this time.

It may be stated, in general, that no evidence of serious injury by alkali was observed on any of the specimens except those of 1:3:6 mix, and that poor fabrication was seemingly responsible for the injury of these specimens.

RESULTS OF TESTS

The tests thus far described are essentially of a chemical nature and serve to indicate, through gain in weight by accumulation of secondary salts, and absorbed moisture, and loss of soluble material, the effect of alkali attack, and the degree of protection offered by the various treatments.

Table 1 gives in condensed form, the results of the tests covering a period of three years, together with the quantities of tar and paraffin absorbed on a percent-by-weight basis. It will be noted that the amount of protectives taken up increases with the leanness of the mix and is proportionate to the number of coats applied. Considering the quantitative effect of alkali attack, indicated in columns 18 and 21 of the table, it will be observed that the total percentage of gain and loss increases with the leanness of the mix and decreases with the quantity of protectives applied.³

An interesting point is the very small percentage of dissolved material (column 21) as compared with the

¹ Protection of Concrete against Alkali, Public Roads, vol. 5, No. 3, May 1924 and another article under the same title in Public Roads, vol. 6, No. 11, January, 1926.

² The solution contained 7.78 grams $MgSO_4$ and 6.61 grams Na_2SO_4 per liter.

³ An apparent exception is indicated in batch 17, column 18, where an excessive gain in weight resulted from partial failure of two of the cylinders.

large increase in secondary salts, chiefly carbonates of lime and magnesia, which exceed $4\frac{1}{2}$ per cent in untreated 1:3:6 specimens stored in alkali (batch 14, column 15). It will be noted that, during the third year, when the samples were partly immersed in the alkali solution (column 12), appreciably lower values for absorbed moisture were obtained than in previous years (columns 10 and 11), while the average quantity of secondary salts remained about constant (column 15).

Regarding the indicated protection afforded by the various treatments, as shown by the gain in weight (column 18) and loss by chemical and mechanical action (column 21), it will be observed that batches receiving a maximum quantity of both tar and paraffin are about equally protected (batches No. 5, 7, 12, 13, 18, and 19), but where the seal coat was omitted

(batches No. 4, 6, 11, and 17) there was a decrease in protection of from one-third to two-thirds in the case of tar-treated samples and about two-fifths for those treated with paraffin.

As stated in a previous report the effect of alkali attack on samples immersed in paraffin solution (batch No. 6) appeared to be mainly superficial. This is further substantiated by a relatively high loss by abrasion and chemical action (0.07 per cent) and low percentage of absorbed moisture (1.12 per cent).

Comparing the chemical effect of alkali attack on the untreated samples after three years' exposure (batches Nos. 1, 8, and 14, columns 18 and 21) with that of cylinders receiving the maximum protective treatment (batches Nos. 5, 7, 12, 13, 18, and 19, columns 18 and 21) the results of the tests indicate for the latter at least a fourfold protection.



FIG. 1.—SURFACE CONDITION OF 1:2:4 CONCRETE CYLINDERS WITH VARIOUS TREATMENTS AFTER THREE YEARS IN ALKALI SOLUTION AND TAP WATER

TABLE 1.—Effect of alkali on treated and untreated concrete

Batch No.	Mix	Treatment	Weight after treatment	Total tar absorbed	Paraffin absorbed	Weight after —			Moisture absorbed after —			Secondary salts formed after —			Total gain in weight after —			Total loss after ¹		
						1 year	2 years	3 years	1 year	2 years	3 years	1 year	2 years	3 years	1 year	2 years	3 years	1 year	2 years	3 years
						Grams	Grams	Grams	Per cent	Per cent	Per cent	Per cent	Per cent	Per cent	Per cent	Per cent	Per cent	Per cent	Per cent	Per cent
1	1:1 $\frac{1}{2}$:3	Untreated, stored in alkali	11,252	1.12	—	11,074	11,105	11,096	0.72	0.77	0.41	2.96	3.18	3.45	3.68	3.95	3.86	0.14	0.16	0.17
2	1:1 $\frac{1}{2}$:3	Untreated, stored in tap water	11,641	0.49	—	11,315	11,322	11,216	.48	.47	.09	1.18	1.27	.70	1.66	1.74	.79	.08	.08	.10
3	1:1 $\frac{1}{2}$:3	6 coats water-gas tar	11,129	.82	—	11,422	11,429	11,381	.18	.27	.01	.68	.64	.47	.86	.91	.48	.01	.02	.04
4	1:1 $\frac{1}{2}$:3	10 coats water-gas tar	11,326	1.12	1.84	11,592	11,463	11,591	.21	.34	.20	.92	.86	.92	1.13	1.26	1.12	.06	.07	.07
5	1:1 $\frac{1}{2}$:3	10 coats water-gas tar, 1 coat coal tar	11,463	—	2.32	11,493	11,524	11,491	.15	.32	.11	.57	.68	.59	.72	1.00	.70	.03	.03	.03
6	1:1 $\frac{1}{2}$:3	24 hours in paraffin solution	10,687	—	—	11,172	11,185	11,164	.92	.60	.54	3.62	4.06	3.93	4.54	4.66	4.47	.17	.19	.27
7	1:1 $\frac{1}{2}$:3	4 coats paraffin	11,304	—	—	11,119	11,178	11,738	.50	.31	.22	2.27	2.50	2.36	2.77	2.81	2.58	.13	.17	.17
8	1:2:4	Untreated, stored in alkali	10,872	.46	—	8,445	8,428	8,405	.36	.30	.13	1.47	1.33	1.23	1.83	1.63	1.36	.04	.07	.07
9	1:2:4	Untreated, stored in tap water	8,293	1.29	—	11,053	11,176	11,158	.36	.23	.09	.93	.90	.86	1.29	1.13	.95	.01	.04	.04
10	1:2:4	6 coats water-gas tar	11,099	1.46	—	11,737	11,724	11,693	.25	.26	.16	.95	.83	.66	1.20	1.09	.82	.03	.03	.03
11	1:2:4	10 coats water-gas tar	10,410	—	2.62	10,902	10,967	10,988	.99	1.15	.96	3.74	4.20	4.59	4.73	5.35	5.55	.24	.26	.33
12	1:2:4	10 coats water-gas tar, 1 coat coal tar	11,099	.64	—	10,831	10,847	11,623	.45	.63	.53	2.66	2.63	3.11	3.11	3.26	3.25	.12	.15	.18
13	1:2:4	24 hours in paraffin solution	10,852	2.26	—	11,099	11,147	11,239	.42	.75	.98	1.85	1.97	2.58	2.27	2.72	3.56	.07	.11	.12
14	1:3:6	Untreated, stored in alkali	10,887	2.94	—	11,051	11,045	11,015	.32	.43	.10	1.20	1.03	1.08	1.52	1.46	1.18	.04	.07	.08
15	1:3:6	Untreated, stored in tap water	11,415	—	2.77	11,612	11,600	11,591	.29	.58	.44	1.39	1.04	1.10	1.68	1.62	1.54	.07	.03	.04
16	1:3:6	6 coats water-gas tar	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—
17	1:3:6	10 coats water-gas tar	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—
18	1:3:6	10 coats water-gas tar, 1 coat coal tar	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—
19	1:3:6	24 hours in paraffin solution, 4 coats of paraffin	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—

¹ Material at bottom of container.

² Three cylinders in batch. All other batches contained 4 cylinders.

An examination of the test specimens after breaking (see fig. 2) indicates a very unequal grading of the coarse aggregate in the tar-treated samples resulting in high porosity and frequently in honeycomb structure. This imperfect grading is especially noticeable in the leaner mixes where partial failure of the test pieces occurred during the third year of exposure to alkali attack (batches No. 16 and 17, columns 18 and 21).

No evidence of alkali attack is noticeable in any of the specimens except possibly in the 1:3:6 mix showing a disproportionate amount of coarse aggregate. It will be observed that the depth of tar penetration is dependent upon the porosity of the concrete and varies from about one-fourth inch in the denser mixes to almost complete saturation in the 1:3:6 mixes receiving 10 coats of tar.

QUALITY OF TARS

Upon completion of the foregoing investigations preliminary tests were made with various types of water-gas and coal tars in regard to their penetration and waterproofing values in order to qualify the material found best suitable for further work. As a result of these tests the following tentative specifications were drawn up.

PROVISIONAL SPECIFICATION FOR TARS FOR USE IN PROTECTION OF CEMENT CONCRETE

I. *Grades.*—The materials covered by this specification shall be supplied in one or both of the following grades, as ordered by _____ (the purchaser).

Grade	Material
TW-1-X	Water-gas tar for absorptive treatment.
TR-1-25	Fluxed refined tar for seal application.

II. *Materials.*—Materials supplied under this specification as—

1. Grade TW-1-X shall be crude water-gas tar, which may be treated for the removal of excess water if necessary to meet the detail requirements of this specification.

2. Grade TR-1-25 shall be prepared from refined gas-house, coke oven, and water-gas tars fluxed with suitable distillates.

III. *General requirements.*—The tars shall be homogeneous.

IV. *Detail requirements.*—The respective grades of tar shall meet the following requirements:

	Grade TW-1-X	Grade TR-1-25
1. Specific gravity, 25°/25° C. (77°/77° F.)	1.030 to 1.100	1.090 to 1.190.
2. Specific viscosity at 40° C. (104° F.)	Not more than 3.0.	35.0 to 60.0.
3. Total distillate by weight:		
To 170° C. (338° F.)		2.0 to 8.0 per cent.
To 235° C. (455° F.)		10.0 to 20.0 per cent.
To 270° C. (518° F.)		18.0 to 30.0 per cent.
To 300° C. (572° F.) not more than	50.0 per cent.	38.0 per cent.
(a) Softening point of residue, not more than		65° C. (149° F.).
4. Bitumen (soluble in carbon disulphide), not less than	98.0 per cent.	80.0 per cent.
5. Water, not more than	3.0 per cent.	2.0 per cent.

V. *Methods of testing.*—Tests of the physical and chemical properties of the tars shall be made in accordance with the following methods:

1. Specific gravity: A. S. T. M. tentative test D70-20T; Proc. A. S. T. M., 1920, Part I, p. 764.

2. Specific viscosity (on first 50 c. c.): United States Department of Agriculture Bulletin 314, p. 7.

3. Distillation test: A. S. T. M. standard method D20-18 (A. S. T. M. Standards, 1924, p. 951).

The following modifications in procedure of the A. S. T. M. method shall be applied:

The distillation test may be made on the sample as received without dehydration, if water is present not to exceed 2 per cent,

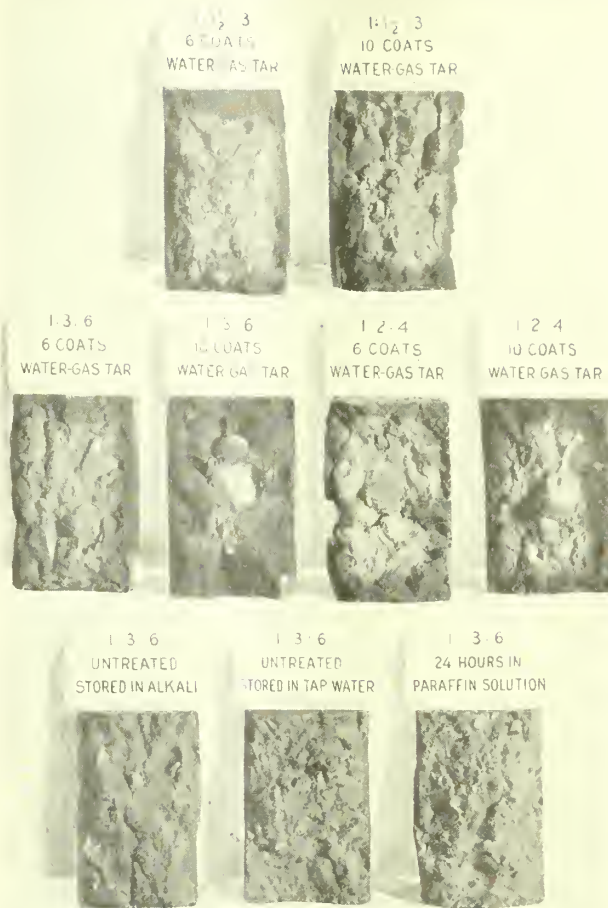


FIG. 2.—CONDITION OF THE INTERIOR OF CONCRETE SPECIMENS AFTER THREE YEARS IN ALKALI SOLUTION AND TAP WATER

but the results shall be reported on a dry basis. (Par. 2 of A. S. T. M. method D20-18.)

The thermometer shall be placed in the flask so that the top of the bulb is level with the lowest point of junction between tubulature and neck of the flask. (Par. 4 of A. S. T. M. method D20-18.)

For use in the distillation test the thermometer shall be calibrated by distilling (a) water, (c) chemically pure naphthalene, and (c) chemically pure diphenylamine in the apparatus as assembled for a test. When the pure material is distilling at the specified rate, and not less than 15 cubic centimeters have been condensed, the thermometer reading and a barometric reading are taken. The actual temperatures are calculated from the equations:

For water, temperature = $100 + 0.037$ (mm. barometer - 760);
For naphthalene, temperature = $218 + 0.058$ (mm. barometer - 760);
For diphenylamine, temperature = $302 + 0.055$ (mm. barometer - 760).

From the observed thermometer readings and corresponding actual temperatures as thus calculated, thermometer readings corresponding to specified fractionating temperatures shall be calculated and used in distillation tests of bituminous materials. (Par. 5 of A. S. T. M. method D20-18.)

The rate of distillation shall be so regulated that 50 to 70 drops pass over every minute. (Par. 6 of A. S. T. M. method D20-18.)

3 (a). Softening point: A. S. T. M. standard method D36-24 (A. S. T. M. Standards, 1924, p. 955).

4. Bitumen soluble in carbon disulphide: A. S. T. M. tentative method D4-23T (Proc. A. S. T. M., 1923, Part I, p. 751).

5. Water: A. S. T. M. standard method D95-24 (A. S. T. M. Standards, 1924, p. 901).

VI. *Notes.*—1. The requirements of this specification for grade TR-1-25 are based on United States Government Master Specification No. 281, Tar for use in repair work.

ABSORPTIVE AND WATERPROOFING PROPERTIES OF TAR

In addition to the experiments thus far recorded it seemed desirable to determine the penetrating and waterproofing properties of water-gas tar when applied to cement mortars and concrete of varying consistencies both by surface application and immersion. The tests were made on 1:1½ and 1:2 mortar and 1:1½:3, 1:2:3, and 1:2:4 concrete cylinders of both wet and dry

consistency, with water-gas and coal tar conforming to provisional specifications TW-1-X and TR-1-25.

The immersion treatment was carried out on cylinders, four to a batch, which, after weighing, were submerged successively in water-gas tar, the first for one-half hour, the second for 1 hour, the third for 1½ hours, and the fourth for 2 hours. After each treatment the samples were allowed to dry out thoroughly

TABLE 2. Absorption of water-gas tar and moisture by mortar and concrete cylinders stored seven days in water and seven days in air

1	2	3	4	5	6	7	8	9	10	11	12	13
Group No.	Sample No.	Mix	Consistency ¹	Tar treatment	Weight before treatment	Tar absorbed		Penetration	Dried	Water absorbed ²		
					Grams	Grams	Per cent	Inch	Hours	Per cent	Per cent	Per cent
1.	1	1:1½	Dry	Immersion, ½ hour	456	3	0.66	1½	1½	1.97	0.43	0.64
	2	1:1½	do	Immersion, 1 hour	455	5	1.10	1½	1	.87	.43	.65
	3	1:1½	do	Immersion, 1½ hours	455	6	1.32	1½	1½	.87	0	.43
	4	1:1½	do	Immersion, 2 hours	454	8	1.76	3½	2	.87	0	0
2.	5	1:1½	do	Painting, 1 coat	459	2	.44	1½	1½			
	6	1:1½	do	Painting, 2 coats	456	3	.66	1½	3¼			
	7	1:1½	do	Painting, 3 coats	459	4	.87	1½	1	2.39	0	0
	8	1:1½	do	Painting, 4 coats	456	5	1.10	1½	1½	1.74	0	0
3.	9	1:1½	Wet	Immersion, ½ hour	430	6	1.39	1½	1½	3.25	.22	.45
	10	1:1½	do	Immersion, 1 hour	430	7	1.63	1½	3¼	1.62	.22	.45
	11	1:1½	do	Immersion, 1½ hours	439	9	2.05	3½	1	.90	0	.22
	12	1:1½	do	Immersion, 2 hours	437	11	2.52	1½	1½	.91	0	.22
4.	13	1:1½	do	Painting, 1 coat	432	2	.46	1½	1½			
	14	1:1½	do	Painting, 2 coats	433	4	.92	1½	1½			
	15	1:1½	do	Painting, 3 coats	432	5	1.16	1½	1½	3.71	0	.45
	16	1:1½	do	Painting, 4 coats	433	6	1.39	1½	1½	3.45	0	.22
5.	17	1:2	Dry	Immersion, ½ hour	459	4	.87	1½	1½	1.74	.21	.43
	18	1:2	do	Immersion, 1 hour	462	5	1.08	1½	1	.86	.21	.22
	19	1:2	do	Immersion, 1½ hours	453	6	1.32	1½	1½	.88	.44	.65
	20	1:2	do	Immersion, 2 hours	452	7	1.55	3½	2	.88	.44	.44
6.	21	1:2	do	Painting, 1 coat	456	2	.44	1½	1½			
	22	1:2	do	Painting, 2 coats	464	3	.65	1½	3¼			
	23	1:2	do	Painting, 3 coats	456	4	.88	1½	1	2.40	.21	.44
	24	1:2	do	Painting, 4 coats	464	5	1.08	1½	1½	.86	0	.21
7.	25	1:2	Wet	Immersion, ½ hour	441	4	.90	1½	1½	2.72	.22	.22
	26	1:2	do	Immersion, 1 hour	433	7	1.62	1½	3¼	.84	.45	.45
	27	1:2	do	Immersion, 1½ hours	441	8	1.81	3½	1	1.12	.22	.22
	28	1:2	do	Immersion, 2 hours	442	9	2.04	3½	1½	1.04	.22	.22
8.	29	1:2	do	Painting, 1 coat	440	2	.45	1½	1½			
	30	1:2	do	Painting, 2 coats	440	4	.91	1½	1½			
	31	1:2	do	Painting, 3 coats	440	5	1.14	1½	1½	3.86	.44	.88
	32	1:2	do	Painting, 4 coats	440	6	1.37	1½	1½	2.06	.22	.22
9.	33	1:1½:3	Dry	Immersion, ½ hour	1,589	18	1.13	1½	1½	2.00	.12	.43
	34	1:1½:3	do	Immersion, 1 hour	1,558	21	1.35	3½	1	1.78	.19	.62
	35	1:1½:3	do	Immersion, 1½ hours	1,575	23	1.46	3½	1½	1.32	.31	.50
	36	1:1½:3	do	Immersion, 2 hours	1,580	24	1.52	3½	2	1.06	.31	.44
10.	37	1:1½:3	do	Painting, 1 coat	1,625	5	.31	1½	1½			
	38	1:1½:3	do	Painting, 2 coats	1,656	8	.48	1½	3¼			
	39	1:1½:3	do	Painting, 3 coats	1,625	10	.62	1½	1	1.90	.06	.54
	40	1:1½:3	do	Painting, 4 coats	1,656	12	.72	1½	1½	.96	.06	.30
11.	41	1:1½:3	Wet	Immersion, ½ hour	1,566	21	1.34	1½	1½	2.54	.18	.44
	42	1:1½:3	do	Immersion, 1 hour	1,648	23	1.40	3½	3¼	1.62	.30	.47
	43	1:1½:3	do	Immersion, 1½ hours	1,585	24	1.51	3½	1	1.20	.12	.44
	44	1:1½:3	do	Immersion, 2 hours	1,585	25	1.58	3½	1½	1.06	.12	.24
12.	45	1:1½:3	do	Painting, 1 coat	1,579	5	.31	1½	1½			
	46	1:1½:3	do	Painting, 2 coats	1,635	9	.43	1½	1½			
	47	1:1½:3	do	Painting, 3 coats	1,579	9	.57	1½	3¼	2.53	.25	.62
	48	1:1½:3	do	Painting, 4 coats	1,635	12	.73	1½	1	2.07	.06	.12
13.	49	1:2:3	Dry	Immersion, ½ hour	1,632	17	1.04	1½	1½	2.01	.12	.46
	50	1:2:3	do	Immersion, 1 hour	1,632	21	1.29	3½	1	1.52	.24	.42
	51	1:2:3	do	Immersion, 1½ hours	1,644	22	1.34	1½	1½	1.10	.18	.42
	52	1:2:3	do	Immersion, 2 hours	1,640	23	1.41	3½	2	1.20	.18	.42
14.	53	1:2:3	do	Painting, 1 coat	1,575	8	.51	1½	1½			
	54	1:2:3	do	Painting, 2 coats	1,581	12	.76	1½	3¼			
	55	1:2:3	do	Painting, 3 coats	1,575	13	.82	1½	1	2.02	.18	.50
	56	1:2:3	do	Painting, 4 coats	1,581	14	.90	1½	1½	1.69	.25	.43
15.	57	1:2:3	Wet	Immersion, ½ hour	1,559	18	1.16	1½	1½	2.42	.12	.36
	58	1:2:3	do	Immersion, 1 hour	1,575	23	1.46	1½	3¼	1.44	.18	.43
	59	1:2:3	do	Immersion, 1½ hours	1,578	26	1.65	3½	1	1.32	.06	.43
	60	1:2:3	do	Immersion, 2 hours	1,625	28	1.72	1½	1½	1.28	.18	.36
16.	61	1:2:3	do	Painting, 1 coat	1,578	8	.51	1½	1½			
	62	1:2:3	do	Painting, 2 coats	1,557	10	.64	1½	1½			
	63	1:2:3	do	Painting, 3 coats	1,578	12	.76	1½	3¼	2.85	.06	.31
	64	1:2:3	do	Painting, 4 coats	1,557	14	.89	1½	1	2.56	.06	.25
17.	65	1:2:4	Dry	Immersion, ½ hour	1,577	22	1.39	3½	1½	2.26	.62	1.16
	66	1:2:4	do	Immersion, 1 hour	1,576	26	1.65	1½	1	1.06	.50	1.12
	67	1:2:4	do	Immersion, 1½ hours	1,652	27	1.63	1½	1½	1.01	.60	1.07
	68	1:2:4	do	Immersion, 2 hours	1,567	28	1.78	1½	2	.76	.20	.40
18.	69	1:3:4	do	Painting, 1 coat	1,590	8	.50	1½	1½			
	70	1:2:4	do	Painting, 2 coats	1,578	11	.70	1½	3¼			
	71	1:2:4	do	Painting, 3 coats	1,590	12	.75	1½	1	2.57	.74	1.23
	72	1:2:4	do	Painting, 4 coats	1,578	14	.88	1½	1½	1.82	.50	.74
19.	73	1:2:4	Wet	Immersion, ½ hour	1,574	24	1.52	1½	1½	2.48	.77	1.41
	74	1:2:4	do	Immersion, 1 hour	1,578	35	2.22	1½	3¼	1.74	.74	1.35
	75	1:2:4	do	Immersion, 1½ hours	1,537	37	2.41	1½	1	1.28	.57	1.20
	76	1:2:4	do	Immersion, 2 hours	1,554	39	2.51	1½	1½	1.52	.13	1.00
20.	77	1:2:4	do	Painting, 1 coat	1,564	8	.51	1½	1½			
	78	1:2:4	do	Painting, 2 coats	1,619	10	.62	1½	1½			
	79	1:2:4	do	Painting, 3 coats	1,564	12	.77	1½	3¼	3.18	1.06	2.06
	80	1:2:1	do	Painting, 4 coats	1,619	14	.87	1½	1	2.26	.60	1.02

¹ A mix of dry consistency gave approximately 1-inch slump, a mix of wet consistency gave approximately a 6-inch slump.² See text.

and the depth of tar penetration and time required for the drying of each specimen recorded. (Table 2, columns 9 and 10.) Gain in weight after each treatment indicated the quantity of tar absorbed. (Table 2, columns 7 and 8.) The same procedure was followed in the surface treatment, except that the samples were painted with 1, 2, 3, and 4 coats of tar.

The results of the tests, given in Table 2, indicate clearly that dry-mixed mortar and concrete (groups Nos. 1, 5, 9, 13, and 17) are less tar absorbent than similar mixes of wet consistency (groups Nos. 3, 7, 11, 15, and 19), and that the rate of penetration as indicated by time of drying, is slower in the dry mixes than in the wet. This may be attributed to less complete hydration in the former with correspondingly lower colloidal content than where a normal quantity of mixing water was used. It will be observed also that the tar absorbed by concrete (groups Nos. 9, 11, 13, 15, 17, and 19) increases in general with the leanness of the mix, as previously noted, evidently due to increased porosity. Comparing the quantity of tar absorbed, after one-half hour immersion by concrete of all mixes (samples Nos. 33, 41, 49, 57, 65, and 73) with samples of similar mixes receiving four coats of water-gas tar (sample Nos. 40, 48, 56, 64, 72, and 80), the tests indicate that appreciably more tar has been taken up by the immersed specimens. This may be explained in part by a relatively rapid evaporation of the lighter tar constituents when painted in hot weather over a large exposed surface area.

Considering the mortar specimens it appears from the test results that the dry mixes (groups Nos. 1, 2, 5, and 6) are about equally absorbent, but when of wet consistency (groups Nos. 3, 4, 7, and 8) the richer mixes (group Nos. 3 and 4) are the more absorbent.

After receiving the water-gas tar treatment all samples were immersed in water for 10 days and their gain in weight determined (column 11). They were then allowed to dry out for 10 days and given one coat of coal tar (TR-1-25) and again immersed in water and reweighed after 10 and 28 day periods. The moisture absorbed is indicated in columns 12 and 13. These results point to the fact that four coats of water-gas tar or immersion for any period up to two hours without an additional seal coat of coal tar are insufficient to greatly retard the ingress of water into concrete under the above test conditions. With the seal coat of coal tar, however, the quantity of moisture absorbed after 28 days averages less than one-half per cent in the richer mixes (group Nos. 1 to 16) and somewhat over 1 per cent in the leaner mixes (group Nos. 17-20).

The results indicate in general that concrete as well as mortar of dry consistency, although less tar absorbent, is fully as moisture resistant as similar mixes of wet consistency. It will be noticed also that the penetration (column 9) is about the same for samples immersed for one-half hour and those receiving four coats of tar while the water absorption (column 13) is somewhat less in the painted samples. It appears from the foregoing that concrete exposed to the action of alkali solutions should not be leaner than a 1:2:4 mix, and should be protected by at least four coats of water-gas tar (equivalent to one-half hour immersion) and one seal coat of heavy coal tar.

FIELD TESTS

Supplementing the laboratory tests it was decided to obtain information regarding the quantity of tar absorbed per square yard of surface (covering capacity) when applied under conditions approximating those met with on structures situated above water level. For this purpose concrete slabs (36 by 36 by 4 inches) were made with Portland cement, and Potomac River sand and gravel in the proportions of 1:1½:3, 1:2:4, and 1:3:6, and cured in the forms for two and seven days under damp burlap. Some of the slabs were painted immediately after stripping the forms while others were allowed to dry out for 7, 21, and 83 days before painting. All slabs were given four coats of water-gas tar at intervals of about one hour, and one coat of coal tar, in some cases immediately following the final coat of water-gas tar and in others 60 days thereafter. The quantity, by weight, of tar absorbed was determined after each application, and the covering capacity calculated from the weight per gallon of the tars employed (water-gas tar 4,087 grams and coal tar 4,738 grams).

TABLE 3.—Water-gas tar and coal tar absorbed by concrete slabs 36 by 36 by 4 inches

1	2	3	4	5	6	7	8	9	10	11
Slab No.	Mix	Curing	Water-gas tar absorbed by coats					Coal tar absorbed (1 coat)	Rate of application (gallons per square yards)	
			1	2	3	4	Total			
			Gms.	Gms.	Gms.	Gms.	Gms.	Gms.	Water-gas tar (4 coats)	Coal tar (1 coat)
1	1:1½:3	48 hours in form.	87	75	60	59	909	125	0.069	0.026
2	1:2:4	do	97	64	60	63		130	0.069	0.028
3	1:3:6	do	105	91	78	70		165	0.084	0.035
4	1:1½:3	48 hours in form.	143	105	92	70	1,302	136	0.101	0.029
5	1:2:4	7 days in air						139	0.102	0.029
6	1:3:6	do						153	0.116	0.032
7	1:1½:3	7 days in form.	73	58	52	50	924	118	0.057	0.025
8	1:2:4	do	100	107	60	50		120	0.079	0.025
9	1:3:6	do	111	109	83	71		137	0.092	0.029
10	1:1½:3	7 days in form.	75	86	63	53	1,163	127	0.068	0.057
11	1:2:4	7 days in air						129	0.098	0.061
12	1:3:6	do						128	0.119	0.061
13	1:1½:3	7 days in form.	80	72	57	38	905	123	0.061	0.019
14	1:2:4	21 days in air						122	0.079	0.048
15	1:3:6	do						125	0.083	0.053
16	1:1½:3	7 days in form.	100	65	52	30	925	189	0.061	0.040
17	1:2:4	83 days in air						195	0.075	0.041
18	1:3:6	do						230	0.091	0.049

1 Coal tar applied 60 days after water-gas tar.

The results given in Table 3 indicate that the quantity of tar absorbed increases with the leanness of the mix regardless of the method of curing and is greatest in the slabs cured for 7 days in air after remaining in the forms either 48 hours or 7 days. (Slabs Nos. 4, 5, 6, and 10, 11 and 12, columns 8 and 10.)

The tests indicate also that approximately equal amounts of tar are absorbed by samples painted immediately after removal of the forms (slabs Nos. 1, 2, and 3 and 7, 8, and 9, column 8), and after curing for 21 and 83 days in air (slabs Nos. 13 to 18, column 8). Estimated on a per-cent-by-weight basis it will be seen that these samples (tar absorbed taken as 100 per cent) have taken up on an average 34.6 per cent less water-

gas tar and 20.8 per cent less coal tar than those cured for 7 days in air. (Slabs Nos. 4, 5, 6, and 10, 11 and 12, column 8.)

The tests indicate a very appreciable increase in coal tar absorption where the slabs are allowed to dry out for 60 days after the water-gas tar treatment. (Slabs Nos. 10 to 15, column 9.) Comparing the figures given in columns 10 and 11 it will be seen that the maximum absorption for coal tar was at the rate of about one-sixteenth gallon per square yard for one coat on 1:2:4 concrete (slab No. 11, column 11), while that for water-gas tar was about one-eighth gallon for four coats on 1:3:6 concrete cured for seven days in the forms and seven days in air (slab No. 12, column 10).

Figure 3 shows slabs painted with four coats of water-gas tar and one coat of coal tar after one year in the open. The marked penetration of the seal coats in the leaner mixes is shown by the faded tones. The leaner mixes received the heaviest seal coat applications but the surface condition indicates that a still heavier application might have been used.



FIG. 3.—SLABS PAINTED WITH FOUR COATS OF WATER-GAS TAR AND ONE COAT OF COAL TAR AFTER ONE YEAR IN THE OPEN. THE MARKED PENETRATION OF THE SEAL COATS IN THE LEANER MIXES IS SHOWN BY THE FADED TONES

APPLICATION OF TAR

Owing to the varying conditions under which the tar may be applied to structures both above and below water level the following directions for application are recommended:

BRUSH TREATMENT

1. *For cast-in-place concrete above water level and for precast concrete units.*—The forms shall be removed and concrete cured as may be directed by the engineer or as may be required by the specifications governing the work. After removal of the forms and curing, the concrete shall be allowed to dry for at least 10 days before the application of the protective coatings. The concrete shall then be thoroughly coated with at least four coats of water-gas tar, grade TW-1-X, applied cold with a brush or spray and each coat shall be absorbed before the succeeding coat is applied. After the absorption of the final coat, a seal coat of coal tar, grade TR 1-25, shall be applied, preferably at a temperature of about 80° F. and thoroughly brushed into all surfaces. The seal coat shall be allowed to dry for at least four days, or as long as necessary to harden properly before handling.

2. *For cast-in-place concrete below water level.*—The concrete shall be cured in the forms for at least 48 hours and allowed to dry for at least five days after stripping the forms. It shall then be thoroughly coated with at least four coats of water-gas tar, grade TW 1 X, applied cold with a brush or spray, allowing each coat to be absorbed before the succeeding one is applied. After the absorption of the final coat a seal coat of coal tar, grade TR-1-25, shall be applied at a temperature of about 80° F. and thoroughly brushed into all surfaces. The seal coat shall be allowed to dry for at least 24 hours before water is permitted to come in contact with it.

IMMERSION TREATMENT

1. *For precast concrete units.*—The forms shall be removed and concrete cured as may be directed by the engineer, or as may be required by the specifications governing the work. After the required curing period the concrete shall be allowed to dry out for at least 10 days before the application of the protective coatings. The precast units shall then be completely immersed in unheated water-gas tar, grade TW-1-X, and allowed to remain in the bath for a period of at least thirty minutes. They shall then be removed from the bath and allowed to drain and dry thoroughly, after which a seal coat of coal tar, grade TW-1-25, shall be applied at a temperature of about 80° F. and thoroughly brushed into all surfaces. The seal coat shall be allowed to dry for at least four days or as long as necessary to harden properly before handling.

STRUCTURES TREATED WITH WATER-GAS AND COAL TARS

Pier of bridge over Arkansas River, Cowley County, Kans.—While the treatment described in the foregoing pages is still in an experimental stage, numerous structures exposed to the action of alkali of sea water have been successfully treated both above and below water line. As a typical example of under-water

treatment may be cited the case of a bridge pier in Cowley County, Kans., reported by T. J. Lough, materials engineer, Bureau of Public Roads, District No. 5. The schedule of treatment as compiled from Mr. Lough's report is given in Table 4.

TABLE 4.—Schedule of treatment of bridge over Arkansas River, Kans.

Date	Treatment	Time required for application	Number of men required for application	Man-hours per 100 square feet	Time allowed for drying	Tar applied on 49.5 square yards	Rate of application per square yard
1926		Hours		H. m.	Hours	Gallons	Gallon
June 28.....	Cofferdam unwatered.				51		
June 30.....	First coat water-gas tar.	2½	1	0 34	3½	4.8	0.097
Do.....	Second coat water-gas tar.	1¼	2	0 34	13¾	3.0	.060
July 1.....	Third coat water-gas tar.	1¼	2	0 34	8	2.2	.044
July 2.....	Fourth coat water-gas tar.	1¼	1	0 17	13½	1.6	.032
July 3.....	One coat coal tar....	1½	2	0 31	24	2.24	.045
Total.....	Five coats.....	7½		2 30	114½	13.84	(1)

¹Water-gas tar 0.233 gallon, coal tar 0.045 gallon.

The purpose of the treatment was to protect the concrete from the action of alkali water of the Arkansas River. (See fig. 4.) The treatment consisted in applying successive coats of water-gas tar (grade TW-1-X) and one coat of coal tar (grade TW-1-25) to pier No. 1 of the bridge, 2.75 feet above and below water level, the treated band having an area of 49.5 square yards.

It was carried out under very favorable weather conditions, temperature ranging from 68° to 96° F. and a bright sun and light breeze persisting throughout the days of treatment.

The concrete was fabricated of one part cement, two parts of fine Arkansas River sand, and approximately three and one-half parts of clean, washed flint gravel, and was two months old when treated.

DESCRIPTION OF WORK

Pumping units.—Two pumping units were employed to unwater the cofferdam. This was to preclude the possibility of water reaching the concrete under treatment should one pump break down. No difficulty was experienced in holding the cofferdam dry.

Condition of concrete.—At the time of applying the initial coat of water-gas tar the concrete appeared to be dry and in a satisfactory condition to receive the tar. The concrete was wire-brushed immediately prior to the application to remove a film of water-deposited impurities.

Rate of absorption.—Where exposed to the sun's rays and the breeze the initial coat penetrated rapidly. On shaded areas the absorption proceeded more slowly. The penetration was not uniform, indicating variations in the density of the concrete. On the web of the pier, the concrete of which had received the most spading, absorption proceeded at the slowest rate.

A decreasing rate of absorption of the successive coats was noted, requiring increasing time intervals between the successive applications. The priming coat had been fully absorbed when the second coat was applied. Small areas had not been fully penetrated when the third coat was applied and slightly larger areas had not been completely penetrated at the time of placing the fourth coat. Considerable areas of the fourth coat were sticky and not entirely absorbed when the coal-tar seal coat was applied, indicating that the saturation point was approached. Postponement of the seal coat application to permit possible additional penetration of the water-gas tar was considered unwarranted.

Application.—No difficulty was experienced in applying the tars with ordinary calcimine brushes. The seal coat was spread out as thin as practicable and was applied cold, heating not being necessary. The proximity of the cofferdam to the concrete with resultant restricted working space somewhat handicapped the workmen in applying the tar, but a workman of average ability should be able to apply a coat of either kind of tar at the rate of 25 square yards per hour.

Penetration.—Immediately prior to the application of the seal coat a number of holes were drilled through the water-gas tar coats and the depth of penetration was found to average one-eighth inch.

Cost.—The cost of the entire operation was as follows: (1) Moving pumps, \$15.83; (2) pumping, \$177.94; (3) cleaning surface, \$5.35; (4) cost of tars, \$5.60; (5)

applying tars, \$10.81; total, \$215.53, or \$0.484 per square foot of surface.

Summary and conclusions.—The following conclusions may be considered as applying to treatment under conditions similar to those described:

1. Duplicate pumping units are essential.
2. Given favorable weather a 48-hour initial drying period for the concrete is sufficient.
3. Rate of absorption is affected both by density of concrete and exposure to sun and wind.
4. As each coat of water-gas tar is less rapidly absorbed than the preceding one, the time intervals between applications should be correspondingly increased, so far as the working day and a reasonable total period of treatment will permit.
5. A workman of average ability should be able to apply a coat of either kind of tar at the rate of 25 square yards per hour.
6. Approximately 2.5 gallons of water-gas tar per 100 square feet of surface are required for four applications; and one application of coal tar in the amount of 0.50 gallon per 100 square feet.
7. The average penetration of the water-gas tar at the time of applying the seal coat will be approximately one-eighth inch.
8. Absorption of more than four coats of water-gas tar is not obtainable without extending the treating period.
9. The concrete must be held in the dry for five days to permit satisfactory treatment.
10. The efficacy of the treatment can only be determined after a considerable lapse of time, and then only by comparing the condition of untreated concrete masonry exposed to the same stream.

Comparing the tar treatment described in the foregoing report with that of the slabs, Table 4 shows that the four coats of water-gas tar were applied to the bridge pier at the rate of about one-fourth gallon per square yard of surface, whereas the slabs of a similar mix cured for one and three months in air absorbed the tar at the rate of about one-thirteenth of a gallon per square yard. (Table 3, slab Nos. 14 and 17.) The coal tar was applied at approximately the same rate on both slabs and pier (about one-twentieth of a gallon per square yard). From these results it is evident that the quantity of tar absorbed by concrete of the same age and consistency may vary considerably, depending upon local weather conditions and manner of application.



FIG. 4. — APPLICATION OF TAR TO PIER OF ARKANSAS RIVER BRIDGE, COWLEY COUNTY, KANS.

Other structures receiving the tar treatment have been reported as follows:

*Abutments and piers of bridge over the Arkansas River, near Ford, Kansas (F. A. P. No. 236-A, Dist. No. 5).—*The area treated was 2,489 square feet on two abutments and six piers, 3 feet above and 3 feet below low water line. After unwatering the cofferdam the masonry was allowed to dry out for four or five days and the area to be treated was thoroughly cleaned with a stiff wire brush. The treatment consisted of six coats of water-gas tar and one coat of coal tar conforming to the provisional specifications of the Bureau of Public Roads. A coat of water-gas tar was applied by going completely around the pier or abutment and ending at the point of beginning. At the completion of the first coat the tar that had first been applied was dry enough to receive the second coat. About two hours elapsed between the application of the second and third coats. The third coat was allowed to dry for four or five hours before placing the fourth coat. After drying out over night the fifth coat was applied and in the afternoon of the second day the sixth coat was put on.

The seal coat of coal tar was usually placed later in the afternoon of the second day, a swab being used instead of the brush used to apply the water-gas tar. It was found that two men could finish a pier or abutment in two days.



FIG. 5.—TREATED CONCRETE PILES, SANTA MARIA RIVER BRIDGE NEAR SANTA MARIA, CALIF.

The rate of application was about one-third gallon of water-gas tar per square yard of surface and slightly over one-tenth gallon of coal tar. The approximate cost of treatment including pumping, labor, fuel, oil and tars was \$540, or about 22 cents per square foot.

*Coast Highway Bridge over North Channel, Santa Maria River, about 2 miles north of Santa Maria, San Luis Obispo Co., Calif. (F. A. P. 25-C).—*The treatment of piles supporting the bridge girders consisted of two coats of water-gas tar applied before driving and two additional coats after driving covering the exposed portion and extending two feet below the normal ground line. The bed of the stream was dry and there was no difficulty in excavating around the piles to a depth of two feet. The application was made with calcimine brushes. A stream of tar from the spout of a watering pot was run along the top surface of the pile as it lay on one of its sides and this was brushed into the top and sides, giving an especially thick coating on the top. The pile was later turned through 180 degrees and the fourth side was treated, as well as the two vertical sides where additional tar could be absorbed. The piles had dried out after curing for more than a month

so the tar absorption was rapid. The tar was used at a rate of $1\frac{1}{3}$ gallons per 100 square feet. The depth of penetration was about one-eighth inch, or as much as might be expected from three or four applications within a few days.

*Reinforced concrete viaduct between Wilmington and Long Beach, Calif., over part of the west basin of Los Angeles harbor.—*The treatment was here given to concrete footings and extended from the ground surface to two feet below permanent low water level. It consisted in applying about six coats of water-gas tar followed by a seal coat of coal-tar pitch, at a cost, including labor, of 43 cents per 100 square feet for the water-gas tar and 96 cents for the coal tar. The concrete was a 1:3:5 mix, and although the penetration of the tar did not exceed one-eighth inch it was felt that an excellent waterproof condition had been obtained.

*St. Marys River Bridge, Camden-Nassau Cos., Fla.-Ga. (F. A. P. 421).—*The parts treated were the backs of abutments from top of footing to top of embankment, the faces of abutments from top of footing to an elevation 3 feet above high tide, the reinforced piers from top of footing to 3 feet above high tide, the undersides of slabs and girders and the sides of girders of the concrete approaches unexposed to view.

The treatment consisted of ten coats of water-gas tar (grade TW-1-X) uniformly applied with a brush in ten separate applications at intervals of 2 hours each, the first application being made immediately upon the removal of the forms, or as soon thereafter as the concrete became sufficiently dry to receive and hold it.

*Morehead City-Beaufort Bridge, Carteret County, N. C. (F. A. P. 64-A Reo.).—*The treatment consisted of four coats of water-gas tar and one seal coat of coal tar on bascule pier shafts and concrete piles.

The concrete of the four pier shafts consisted of one part cement, two parts of clean bank sand and four parts of washed flint gravel, graded $\frac{1}{4}$ to $1\frac{1}{2}$ inches. The piles were fabricated of one part cement, one and one-half parts sand and three parts gravel, graded $\frac{1}{4}$ to $\frac{3}{4}$ inch in size.

The treatment of the shafts extended from 6 feet above to 6 feet below mean water level. The tars were applied very effectively with calcimine brushes with very long handles as the cofferdams were too close to the shafts to permit free working conditions around the pier.

The concrete was dry and clean when the tar was applied, and the rate of absorption was reasonably uniform except where the concrete had taken up resin from the boards. These places stood out prominently and the absorption was very slow.

Each successive coat showed decreasing absorption and thus required longer intervals between applications, but in no case was the time required for the penetration of a coat more than one hour and 15 minutes. Small areas were not completely dry when the fourth and fifth coats were applied.

Under conditions similar to the above one man should apply 270 square feet per hour of the first coat, 315 square feet per hour of the second, third, and fourth coats, and 210 square feet per hour of the seal coat.

The quantity of tar used on this project per 100 square feet of surface was 2.95 gallons of water-gas tar and 1.10 gallons of coal tar. The method of treatment appears satisfactory and is recommended.

(Continued on page 119)

STATIC AND IMPACT LOADS TRANSMITTED TO CULVERTS¹

DIGEST OF REPORT ON EXPERIMENTAL DETERMINATIONS AT IOWA EXPERIMENT STATION

Experimental determinations of static and impact loads on highway culverts have resulted in data which can be directly applied in culvert design. The experiments have been carried on at the Iowa Engineering Experiment Station in cooperation with the United States Bureau of Public Roads. They have shown that a formula to determine the pressure on a culvert resulting from a concentrated load on the fill above it can be used to determine the maximum obtainable transmitted load. For fills of 5 feet and less the effect of concentrated superimposed loads is appreciable, but for fills greater than 5 feet and widths of culverts not greater than 3½ feet the effect of ordinary truck loadings may be ignored. Moving loads may result in a pressure on the culvert much greater than similar static loads. It is believed that an increase of 50 to 100 per cent over the calculated static load effect is sufficient to take care of the impact produced by moving trucks up to speeds of 10 miles per hour.

The general program of culvert investigations has included work with earth embankments and the theory of loads on culverts which has been or will be reported in experiment station bulletins.²

LAW GOVERNING PRESSURE ON CULVERT FROM CONCENTRATED LOAD AT TOP OF FILL INVESTIGATED

The work with static loads was an attempt to discover the law governing the transmission of loads to culvert tops from static concentrated loads, such as wheel loads when the wheels are applied on the surface of the fill at a distance, H , above the top surface of the culvert. So far as known J. H. Griffith was the first to suggest³ that the formula, $P = \frac{3}{2\pi} \frac{H^3}{H_s^5} T$ (see below for notation) would apply to masses of earth as in this case. This formula was derived long ago by Boussinesq⁴ in his theoretical work on stresses in elastic solids. In the case of an infinite elastic solid bounded by a plane he worked out the effect of a concentrated normal force applied at a point on the surface, calculating the stress distribution and displacements at different points in the solid.

The formula shown is a part of the solution of the general problem of finding the stresses due to a force T applied normal to an infinite plane bounding an elastic solid. The general formula is:

$$W_t = \frac{C_t T}{l}, \text{ in which } C_t = \frac{\Sigma P a}{T} \text{ and } P = \frac{3}{2\pi} \frac{H^3}{H_s^5} T, \text{ where}$$

T = Total concentrated load applied at one point on the surface of the embankment.

W_t = The total corresponding load, per unit of length, transmitted to the top area of the test section.

l = Length of the test section of culvert.

C_t = A calculated coefficient of transmitted load.

Σ = Summation of pressure transmitted to each small unit area of the top of the test section of culvert.

P = The intensity of vertical pressure per unit of area due to T at the center of any small unit area, a of the top area of the test section.

a = Area of each of the small equal units into which the horizontal top area of the test section of the culvert must be divided for summation purposes.

H = Height of embankment above top of the culvert.

H_s = Slant height direct from the point at which T is applied to the center of the particular unit area a for which calculation is being made.

The work with static concentrated loads was confined to determining the applicability of these formulas which were found to apply with a satisfactory degree of closeness.

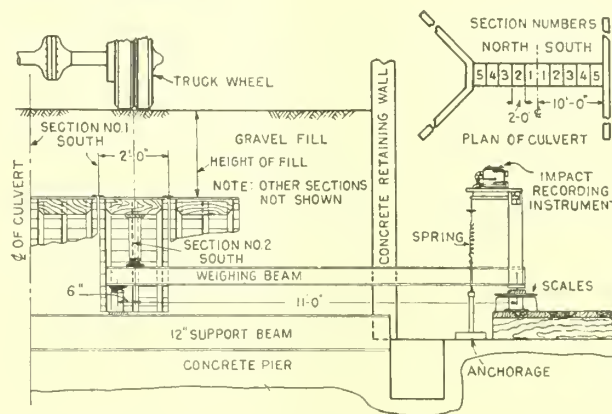


FIG. 1.—SHOWING ARRANGEMENT OF APPARATUS FOR MEASURING PRESSURE ON CULVERT. THE IMPACT RECORDING INSTRUMENT AND THE SPRING ATTACHED TO BALANCE BEAM WAS USED ONLY IN THE IMPACT TESTS

Figure 1 shows the general arrangement of apparatus used in the test. Culvert pipe sections 2 feet in length and 3½ feet outside diameter were placed beneath a gravel fill weighing from 125 to 140 pounds per cubic foot. This gravel was used as taken from the pit and contained a high percentage of clay. A sieve analysis showed about 30 per cent retained on a ¼-inch sieve. Pipe sections No. 2 north and No. 2 south (see fig. 1) were selected for measurement of transmitted wheel loads and each equipped with a lever and platform scale weighing device. The weight of a culvert section is transmitted to the weighing beam 6 inches from the fixed-end knife-edge support. The other end of the weighing beam rests on platform scales 11 feet from the fixed end. Differences in scale readings due to applied wheel loads were therefore multiplied by 23 to give the difference in load on the culvert section. A 3-ton truck loaded with various amounts of gravel was used for loading. The axle spacing of this truck was 13½ feet and the wheel gage 5 feet 10½ inches. The tire equipment consisted of 40 by 6 inch single-tread, solid tires on the front wheels and 40 by 6 inch dual-tread solid tires on the rear wheels.

¹ The full report is contained in Iowa Engineering Experiment Station Bul. 79, Experimental Determinations of Static and Impact Loads Transmitted to Culverts, by M. G. Spangler, Clyde Mason and Robley Winfrey.

² Buls. 31, 36, 47, 57 and 76, Iowa Engr. Exper. Sta.

³ Report of subcommittee on bearing values of soils, Proc. Am. S. of C. E. Aug., 1920, vol. 49, p. 931.

⁴ "Application des Potentiels," by M. J. Boussinesq, Paris, 1885, pp. 65-66.

FIVE SERIES OF STATIC TESTS MADE

The field work with static loads is classified into series A, B, C, D, and E. Each series represents data taken under different conditions of fill, apparatus, and time intervals. Series A, B, and C were run with the embankment material wholly in contact with the pipe, so that the arrangement was similar to a field installation of a circular culvert having the same projection condition. In series D and E this arrangement was modified, as explained later.

Static load series A.—Series A is a set of readings taken with heights of fill on the culvert of 24, 18, 12, and 6 inches with the truck body empty and consequently a relatively light load. The truck was run onto the culvert and stopped with one rear wheel directly over section 2, south, standing in this position for a few minutes only. This placed the other rear wheel within 1.5 inches of being directly over the center of section 2, north. The scale readings for both sections were noted while the truck stood over the culvert. These scales were read again after the truck was removed and the additional load on the 2-foot section calculated.

Static load series B.—The readings for series B were obtained in exactly the same manner as those of series A, except that a heavier load was used on the truck and the heights of fill ranged upward in the order of 6, 12, 18, and 24 inches.

Static load series C.—For each of the runs in series A and B the truck was allowed to stand over the culvert for only the few minutes necessary to take the scale readings. In order that the effect of time on the transmitted load might be observed, series C was conducted, in which the truck was placed over the culvert and allowed to stand over night, or about 15 hours. This series was run in connection with the impact work; that is, impact runs were made during the day and the truck allowed to stand on the culvert at night for observation of static load distribution.

Static load series D.—As indicated above, Series A, B, and C were conducted, using a circular culvert section. After these were completed it seemed desirable to study the load on a section having a flat top which would fulfill the conditions encountered in the case of a rectangular box culvert where the fill is of equal depth at all points. Accordingly, a platform of the same area as the projection of the circular culvert section on a horizontal plane was constructed and made to rest upon the circular section.

Static load series E.—Following series D the gravel fill was removed down to the top of the platform and an attempt was made to calibrate the apparatus by placing weights on the flat top and noting the readings on the scales. It was found during this work that when the gravel fill was in contact with the sides of the platform placed on top of the culvert it was not possible to check the calculated mechanical advantage of the lever system. When the gravel was removed to a depth equal to nine-tenths of the vertical diameter, however, this mechanical advantage ratio could be checked very closely. This fact indicated that the gravel in contact with the culvert section actually helped to support it when a superimposed load was applied, and that the total load transmitted to the flat platform was not recorded on the scale.

To eliminate this probable friction support, a crib was designed and built which prevented the gravel from

coming in contact with the sides of the culvert. This crib consisted of 2 by 6 inch planking placed parallel to the longitudinal axis of the culvert and anchored back into the fill in such manner as to be free to move vertically and still not come in contact with the culvert. The planks were spaced vertically with about 1 inch between them and these openings flashed with tin. Figure 2 shows a sketch of this arrangement.

Series E was run, using the above apparatus and was carried up to the 6-foot level beginning at the 6-inch level. The truck was allowed to stand on the section for a few minutes only. In the earlier series considerable trouble was experienced in maintaining the exact height from the truck tire to the culvert surface which was desired. That is, if readings were being taken at the 12-inch level, the tires would cut into the gravel to such an extent that this distance might be reduced as much as 25 per cent. Since the vertical distance from tire to top of culvert is an important factor affecting the distribution of the load, a reading was taken by means of an engineer's level and rod on the truck wheel for each run of series E, so that this vertical distance was measured exactly.

During the progress of this series, difficulty was encountered at about the 3-foot level, which necessitated removing gravel down to the platform and then replacing it before continuing the tests. The opening

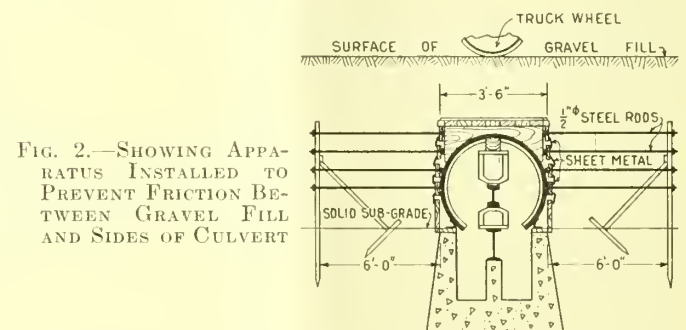


FIG. 2.—SHOWING APPARATUS INSTALLED TO PREVENT FRICTION BETWEEN GRAVEL FILL AND SIDES OF CULVERT

between the bottom of the platform and the top of the cribbing had been flashed with canvas instead of tin. As the fill increased and the crib planks gradually settled downward, the horizontal pressure of the gravel forced the canvas to bulge in and to afford some support to the platform. With this condition the total load being received by the platform was not being indicated on the scales, so the rib flashings were changed to tin before readings were continued. This condition undoubtedly existed to a much less degree at the levels of 2 feet and below, but the effect of load transmitted fell off rapidly at the 3-foot level so that an examination of the apparatus was made and the above trouble discovered.

Table 1 shows the distribution of truck weights for the static load series.

TABLE 1. —Distribution of truck weights for static load series

Static series	Height of fill	Gross weight	Weight on rear wheels	Weight on front wheels
	Feet	Pounds	Pounds	Pounds
A.....	0.5-2.0	11,010	6,590	4,420
B.....	.5-2.0	13,380	8,700	4,680
C.....	2.0-6.0	16,160	11,290	4,870
D.....	.5-2.0	10,570	6,320	4,250
E.....	3.0-6.0	11,010	6,590	4,420

TABLE 2.—Results of static load tests

Series	Height of fill	Weight on one rear wheel	Section 2, south		Section 2, north	
			Load on culvert section ¹	Percentage of wheel load	Load on culvert section ¹	Percentage of wheel load
	<i>Feet</i>	<i>Pounds</i>	<i>Pounds</i>	<i>Per cent</i>	<i>Pounds</i>	<i>Per cent</i>
A	2	3,295	695	21.06	734	22.24
	1½		1,029	31.12	922	27.97
	1		1,465	44.47	1,504	45.62
B	1½	4,350	1,976	60.08	1,798	54.36
	1		2,721	62.55	2,762	63.53
	½		2,045	47.02	1,923	44.23
C ²	2	5,645	1,309	30.09	1,398	32.15
	1½		975	22.44	925	21.24
	1		1,127	20.0	1,449	25.7
D	½	3,295	805	14.3	713	12.6
	1		736	13.1		
	1½		529	9.8	368	6.5
E	2	3,160	644	11.4	506	9.0
	1½		2,339	71.0		
	1		1,579	48.3		
	½	3,295	1,044	31.7		
	1		561	17.1		
	1½		2,926	92.7		
	2	3,295	2,491	78.8		
	1½		1,748	55.3		
	1		1,217	38.5		
	½	3,295	590	17.9		
	1		432	13.2		
	1½		209	6.4		
	2	3,295	200	6.1		
	1½					
	1					

¹ The values given represent the average of 10 or more measurements in every case except series C, where only one measurement is represented.

² Load applied for about 15 hours in every case except for section 2, north, with a 6-foot fill, which was for 45 hours.

EXPERIMENTAL RESULTS COMPARED WITH THEORETICAL LOADS

Figure 3 shows curves plotted according to the formula, $C_t (\%) = a \Sigma \frac{3 H^3}{2 \pi H_s^5} \times 100$ (giving the values obtained for C_t expressed as a percentage of the applied load). The load is figured for a top area of 3.5 by 2 feet, which equals the horizontal projection of the test section. The various heights of fill are plotted as ordinates and the percentages of applied load transmitted to the culvert section as abscissæ. The drawings also show the relation between the percentage of the load applied to the culvert and height of fill as indicated by test results. In series A, B, and D, it will be seen that the actual observed percentages of the applied load distributed to the culvert section fall materially short and in series C generally somewhat short of the theoretical curve, especially below the 3-foot level. In series E, however, when an attempt was made to eliminate side support of the section from the surrounding gravel, the observed percentages come very close to the theoretical curve, sometimes reaching, but practically never exceeding it. The conditions existing in series E were such that the observed loads may be said to be identical with those obtained under field conditions, where the top slab of the culvert receives the total load imparted to the structure by a concentrated load on top of the fill.

CONCLUSIONS REGARDING STATIC LOAD RESULTS

The theoretical formula seems to give a locus showing the maximum possible percentage of load transmitted through any thickness of fill. In the experimental work, however, this maximum load generally was not reached, but when conditions were most favorable, as in series E, the experimental results came very close to the theoretical.

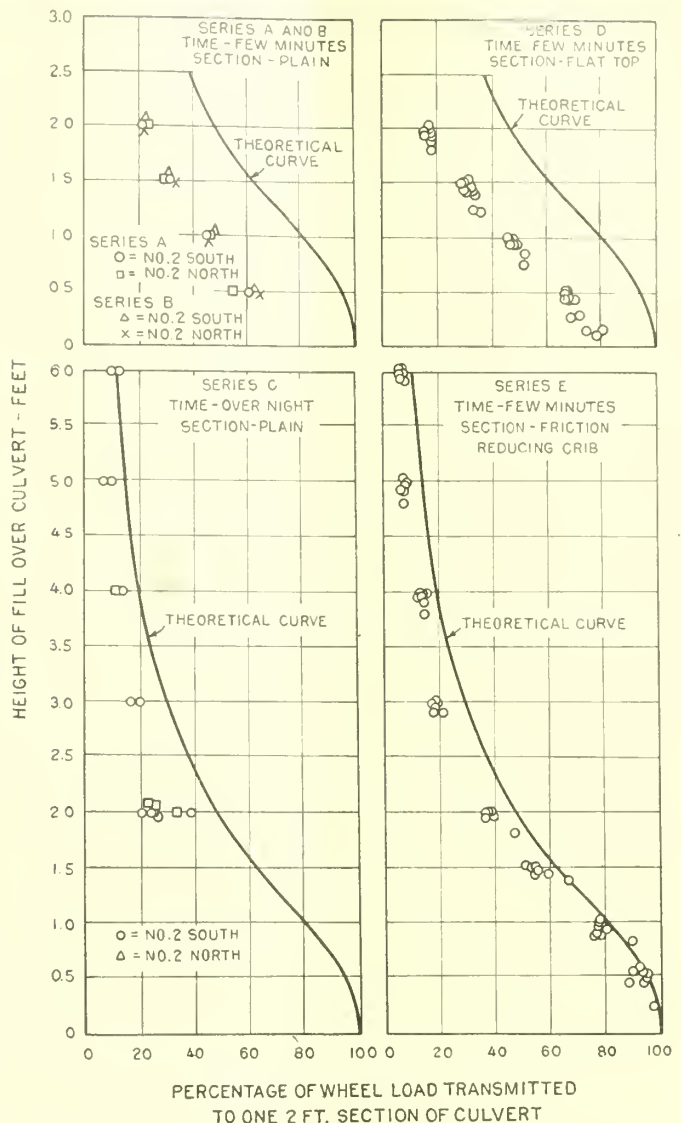


FIG. 3.—COMPARISON OF EXPERIMENTAL RESULTS FROM STATIC LOAD TESTS WITH THEORETICAL CURVE

It is rather remarkable that the theory developed by Boussinesq for elastic solids fits so closely when applied to a granular material such as gravel. The observed points on the curves in Figure 3 were taken under track conditions varying from the loose freshly filled embankment to a wheel track worn fairly hard and smooth by the truck. Under certain conditions, as in series E, the load on the culvert section approaches close to the theoretical curve.

The formula $W_t = \frac{C_t T}{l}$ should be used in designing culverts or conduits through embankments when it is necessary to design for superimposed concentrated loads of a static nature. For convenience in using the formula the values of C_t have been computed for the values of H of 0.5, 1.0, 1.5, 2.0, 3.0, 4.0, 5.0, 7.0, 10.0, 15.0, and 20.0 feet for any size structure up to 10 by 20 feet and are to be published later.

Table 3 gives such values for 2-foot section of a culvert 3½ feet wide.

TABLE 3. Values of the coefficient C_i for a section 3.5 feet wide and 2 feet long

Height of fill in feet	Values of C_i		
	Rear wheel directly over center of area	Other rear wheel (5.875 feet away)	Total ¹
0.5	0.95262	0.00005	0.95267
1	.79758	.00042	.79800
1.5	.61295	.00131	.61426
2	.46271	.00549	.46820
3	.27283	.00679	.27962
4	.17351	.01107	.18458
5	.11822	.01435	.13257
6	.08510	.01628	.10138
7	.06393	.01700	.08093
8	.04968	.01689	.06657
9	.03965	.01623	.05589
10	.03236	.01530	.04766
11			
12			
15	.01464	.01015	.02479
20	.00829	.00670	.01499

¹ The effect of the front wheels is negligible in this case.

For culverts up to $3\frac{1}{2}$ feet in width and under 5 feet or more of embankment the effect of a concentrated load at the surface is negligible compared with the weight of the fill. For greater widths of culvert the depth of embankment at which concentrated loads may be neglected is greater than 5 feet, but will be moderate.

APPARATUS FOR IMPACT MEASUREMENTS

The same general arrangement of apparatus and material as used for the static tests was used for the impact measurements. These measurements were made on section 2, south as illustrated in Figure 1. This section was chosen because when the truck passed through the center of the culvert, one wheel passed over the center of the section.

Additional apparatus was required for measuring the rapid movement of the balance beam under impact. A specially designed instrument for this purpose was mounted on the wooden guard above the balance beam. This instrument consisted of a pointer equipped with a pen which traced its movement on a continuous sheet of recording paper. The pointer was actuated by a system of levers attached to a vertical rod which rested on top of the balance beam of the scale near its end. The continuous recording paper was moved by a 6-volt motor. In addition to the pointer which recorded movement of the balance beam, two other pointers with pens attached, were used, one to record half seconds of time at the bottom of the recording paper and the other to indicate, at the top of the paper, revolutions of a drum of known diameter which unreeled as the truck moved forward. From these last two records the speed of the truck as it passed over the culvert could be calculated.

The movement of the balance beam was retarded by means of a spiral spring attached to the bottom of the scale pan and anchored to the floor of the scale house. Springs of various sizes and strengths were used according to the amount of impact so that the movement of the pointer arm could be held down within the limits of the recording paper. Weights on the scale pan and on the scale platform were also used to retard the movement of the pointer. These various combinations were calibrated and treated as separate apparatus.

The operation of the above described apparatus was as follows: As the truck moved toward the culvert it unwound a small brass wire from a drum placed at the starting point whose diameter was exactly 4 inches. Each revolution of the drum made an electrical contact

which actuated the pointer at the top of the recording sheet and thus a record was obtained of the distance the truck moved for any half second. As the truck moved across the culvert, the effect of the rear wheel was transmitted through the gravel fill to the culvert section. This transmitted effect was indicated by movement of the beam of the scale on which the weighing lever for this particular section rested. The movement of the scale beam was recorded by the pointer of the impact instrument and the highest peak of the ink trace was called the impact reading. The truck was then backed onto the culvert and the position of the pointer noted while the truck was standing still. This was called the static reading. The difference between these two readings formed the basis of calculating the increase in effect on the culvert due to a moving load over the effect of an equal static load.

IMPACT MEASURING DEVICE CALIBRATED

In setting up an apparatus to determine calibration curves for the impact data, resort was made to the well-known physical law that a weight suddenly applied to a surface, but without a blow, produces a maximum pressure on the receiving surface equal to twice the applied weight, but continuing only an infinitely short time. An instantly applied load, then, momentarily produces twice the ultimate continuous effect of a slowly applied load. With this law in mind, an apparatus was planned by which loads of various magnitudes could be suddenly applied to the culvert section over which the original impact runs were made.

To make the calibration tests, the gravel fill was removed from the culvert section and a wooden framework built over and supported by the culvert section. This framework was so arranged that it could be loaded to give a pressure on the culvert equal to that of any desired height of fill. At the center of this framework and directly over the culvert a cage containing any desired weight was suspended so as to just touch the platform or framework. By means of a trip and rope this load could be suddenly applied without any appreciable fall. A record was made of the resulting deflection of the system of levers at the instant of application and also as a static load. Data were thus secured for the preparation of calibration curves representing various heights of fill and other test conditions. Figure 4 shows a typical calibration curve.

These curves were used to determine the suddenly applied load which would produce the same deflection of the recording apparatus as caused by any particular truck run. Knowing the deflection caused by truck impact the proper calibration curve gives the suddenly applied load which causes the same effect. This suddenly applied load is multiplied by two to give the static load which would cause the same effect continuously. This value is then compared with the calculated theoretical static load effect and the percentage of momentary impact determined.

Take for example, a typical truck run over a 3-foot fill. The initial reading on the recording paper was 73, the static reading was 97, and the impact reading as the wheel hit the bottom of the hole in the track was 118. This gives an impact difference of 45 and a static difference of 24. From the proper calibration curve (fig. 4) 850 pounds is found as the load which if applied suddenly produces the same throw of the scale beam. Doubling this to get an equivalent static load gives 1,700 pounds. The actual static load is 960 pounds as shown by the lower or static load calibration curve.

Seventeen hundred pounds is 108 per cent of 1,581 pounds, the theoretical transmitted static load under the same conditions. The moving truck, therefore, produced a momentary effect on the culvert equal to 108 per cent of the theoretical effect of the truck when standing over the culvert.

McCULLUM-PETERS STRAIN TELEMETER USED

One group of the impact tests was conducted using the McCullum-Peters strain telemeter. This instrument is designed to measure stresses in structural members by means of a recording device which registers the strain of the member under stress. There are 12 gauges in connection with the instrument so that it is capable of registering strains at 12 points in a member simultaneously. Each gauge consists of two hardened points exactly 8 inches apart. These points are clamped to the structural member at the place where observations are desired. As the member is lengthened

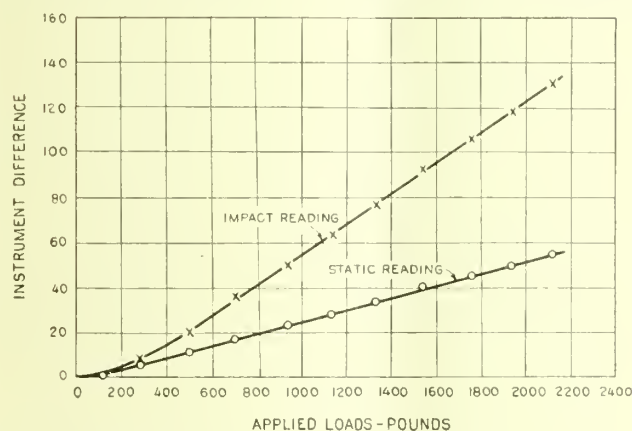


FIG. 4.—SAMPLE CALIBRATION CURVE USED IN IMPACT TEST

or shortened under stress, depending on whether the member is in tension or compression, the points cause a change in the electrical carrying capacity of a carbon resistance unit in the gauge and the amount of current passing through the carbon resistance unit is thereby varied. This variation in current causes a small mirror at the main recording unit of the instrument to revolve slightly and throw a beam of light so that it describes an arc which is proportional to the strain in the structural member. The movement of this beam of light is recorded on a roll of sensitized paper which moves at a constant speed and a photographic record is thereby made of the change in strain of the member at each point where a gauge is attached. Since strain is proportional to the load applied to a beam, other conditions being the same, a record showing changes in strain will also be a record of change in load applied to the beam.

In applying the above instrument to the culvert section on which impact observations were made, the platform scale on which the end of the weighing lever rested was removed and an unyielding support substituted therefor. This produced a beam simply supported at each end with a concentrated load applied eccentrically. Ten of the gauges were clamped to the lower flange of this beam, arranged in pairs. Then a photographic record of the change in stress in the beam was made by taking an exposure first, with no load on the culvert (except the gravel fill), second, at the time of impact, and third, with the truck standing over the culvert. Although 10 gauges were used in the

impact runs, only the two which were nearest the knife edge supporting the culvert were properly calibrated. The calibration was by the same general method as used with the other impact measuring apparatus supplemented by laboratory calibrations by independent methods. The results given are based on the strains recorded by only two gauges. Due to the short time which this instrument was available, observations were made only on the 2 and 3-foot levels.

IMPACT TESTS MADE UNDER DIFFERENT CONDITIONS

The impact tests runs were made with a 3-ton truck loaded with various amounts of gravel and with tire equipment as previously described. Table 4 shows the heights of fill and wheel loads for the four groups of impact tests.

TABLE 4.—Truck loads for impact groups

Group No.	Heights of fill	Gross load	Weight on rear wheels	Weight on front wheels
	<i>Feet</i>	<i>Pounds</i>	<i>Pounds</i>	<i>Pounds</i>
1.	2, 3, 4, 5	16, 180	11, 400	4, 780
2.	6, 8, 10	19, 030	13, 660	5, 370
3.	2, 3, 4, 5	31, 940	25, 700	6, 240
4.	2, 3, 4, 5	16, 160	11, 290	4, 870
	2, 3	17, 750	12, 600	5, 150
	2, 3	15, 960	11, 020	4, 940

It was found that with a smooth track over the culvert, the truck produced only a quite small percentage of impact. Since in actual service, the roadway over a culvert is quite often rough and full of ruts, it was deemed desirable to place obstructions in the track so that this naturally rough condition might be simulated. The truck was allowed to pass over these obstructions in such a manner that after leaving the obstruction the wheel struck the track directly over the center of the culvert section. In the case where a hole was dug in the track to produce impact the hole was centered directly over the section. The obstructions used in these impact runs were a 2 by 4 inch block of wood, a 4 by 6 inch block, a 2-inch hole dug in the wheel track, a 4-inch hole and a 6-inch hole.

FOUR GROUPS OF TESTS MADE

The prosecution of the impact phase of load determination was spread out over a long period of time and conducted with different men in charge of the work. These conditions produced data of such a nature that it is convenient to divide the whole study into four distinct phases, or groups, which have been designated as groups 1, 2, 3, and 4.

Impact Group 1.—Group 1 includes the data taken prior to September 1, 1923, by Clyde Mason. The early work included in this group was, of necessity, of a pioneer nature. So far as could be ascertained, no similar work had ever been performed and it was necessary to do a great deal of experimenting with apparatus before a satisfactory method of recording impact was developed.

Impact Group 2.—Group 2 includes the work done in the spring of 1924 up to about the 1st of July. Impact runs were made on levels of fill from 2 to 6 feet high under the same conditions as those existing when the data for Group 1 was taken; Group 2 constitutes a check series for Group 1.

Impact Group 3.—Group 3 is a set of readings taken on the same levels and under the same conditions as Group 2, except that the culvert was surrounded by the

friction reducing crib mentioned under the discussion of static loads.

Impact Group 4.—Group 4 covers a series of impact observations taken on the 2 and 3 foot levels, in which impact was measured by means of the McCollum-Peters strain telemeter. The method of recording impact by this instrument is of such a different nature from that employed in the other three groups, that these results form a valuable check on the first work.

Table 5 gives the results from two runs selected at random from a large number of tests made. With regard to the complete data included in the report it is noted that the percentages of impact are far from being uniform; that is, two runs apparently made under the same conditions of track, of speed, and apparatus, quite often show radically different results. There are undoubtedly many reasons why these inconsistencies exist. One contributing factor was the difficulty of always guiding the truck so the rear wheel would strike the center of the section. The shifting character of the gravel fill and the steep downward approach to the culvert gave conditions which probably affected the results somewhat. It was also difficult to place the plank obstruction so that the truck wheel would strike the track in the same place each time. In some cases the wheel would merely roll off the edge of the planking and deliver a comparatively light blow to the gravel, while at other times the wheel would follow the path of a projectile and strike the track several inches in front of its customary place, but with a much more severe impact. There seemed to be no way of controlling this, as speed did not seem to cause the difference. In fact, some of the lowest impact percentages were obtained at the highest speeds. In general, the impact seemed to increase as the speed of the truck increased, though the range of speeds was too small to warrant definite conclusions on this point.

TABLE 5.—Results of impact test runs selected at random from each group of tests

Height of fill (feet)	Truck speed (m. p. h.)	Obstacle	Calculated transmitted static load (pounds)	Loads from calibration curve (pounds)		Impact effect, percentage of calculated transmitted static load ¹
				Static	Impact effect	
3	2.3	Smooth track	1,596	970	1,050	66
	2.0	do	1,596	970	1,050	66
	8.1	2 by 4 inch block	1,596	970	1,860	117
	7.4	do	1,596	970	1,720	108
	4.1	4 by 6 inch block	1,596	400	3,160	198
	5.0	do	1,596	970	3,040	190
	7.7	4-inch hole in track	1,596	1,160	3,440	216
	7.0	do	1,596	1,350	3,580	224
	4.1	6-inch hole in track	1,596	1,040	3,080	193
	4.4	do	1,596	620	3,080	193
4	6.1	Smooth track	1,049	1,570	2,100	208
	5.1	do	1,049	1,420	2,040	114
	6.5	4 by 6 inch block	1,049	1,500	2,520	240
	7.2	do	1,049	1,460	2,580	246
	5.8	4-inch hole in track	1,049	1,570	3,840	366
	5.1	do	1,049	1,460	3,580	341
	5.7	6-inch hole in track	1,049	720	2,020	193
	4.3	do	1,049	720	2,160	206
	9.3	Smooth track	908	1,250	1,600	176
	8.6	do	908	1,120	1,360	150
5	8.6	4 by 6 inch block	908	1,120	1,800	198
	9.1	do	908	1,250	1,600	176
	5.0	4-inch hole in track	908	1,000	2,140	236
	6.0	do	908	1,000	2,480	273

¹ Total effect transmitted to culvert top from below on roadway surface.

TABLE 5.—Results of impact test runs selected at random from each group of tests—Continued

GROUP 2						
Height of fill (feet)	Truck speed (m. p. h.)	Obstacle	Calculated transmitted static load (pounds)	Loads from calibration curve (pounds)		Impact effect, percentage of calculated transmitted static load
				Static	Impact effect	
2	2.0	Smooth track	2,642	1,200	1,420	54
	1.7	do	2,642	1,200	1,260	48
	2.5	2 by 4 inch block	2,642	1,200	2,080	79
	2.8	do	2,642	1,320	2,240	85
	1.9	4 by 6 inch block	2,642	1,400	2,560	97
	1.9	do	2,642	1,460	2,340	89
	1.9	2-inch hole in track	2,642	1,540	2,340	89
	3.6	do	2,642	1,590	2,620	99
	2.7	Smooth track	1,581	860	940	59
	2.0	do	1,581	860	1,060	67
3	2.1	2 by 4 inch block	1,581	800	1,580	100
	1.1	do	1,581	800	1,400	89
	1.1	4 by 6 inch block	1,581	850	1,420	90
	2.5	do	1,581	880	1,800	114
	2.1	2-inch hole in track	1,581	880	1,880	119
	5.7	do	1,581	1,000	2,040	129
	2.1	4-inch hole in track	1,581	960	1,700	108
	2.2	do	1,581	960	1,820	115
	3.2	6-inch hole in track	1,581	1,150	2,720	172
	2.0	do	1,581	1,040	2,140	135
4	1.4	Smooth track	1,039	670	860	83
	2.1	do	1,039	720	900	87
	3.8	4 by 6 inch block	1,039	720	1,420	137
	4.3	do	1,039	720	1,340	129
	1.4	4-inch hole in track	1,039	720	1,140	101
	1.4	do	1,039	720	1,320	127
	2.0	6-inch hole in track	1,039	720	1,860	179
	2.4	do	1,039	800	1,820	175
	3.4	Smooth track	751	300	600	80
	3.7	do	751	320	640	85
5	3.3	4 by 6 inch block	751	360	980	131
	3.3	do	751	340	920	123
	3.1	4-inch hole in track	751	340	860	115
	2.1	do	751	340	780	104
	3.7	6-inch hole in track	751	360	1,200	160
	3.7	do	751	410	1,180	157
GROUP 3						
2	3.7	Smooth track	2,948	2,160	2,380	81
	3.7	do	2,948	2,160	3,260	111
	2.2	2 by 4 inch block	2,948	2,070	3,200	108
	1.9	do	2,948	1,980	2,880	98
	6.0	4 by 6 inch block	2,948	2,160	4,240	144
	7.0	do	2,948	2,160	4,220	143
3	1.4	Smooth track	1,764	680	780	44
	3.4	do	1,764	680	800	45
	2.7	2 by 6 inch block	1,764	600	980	56
	2.4	do	1,764	550	940	53
	3.5	4 by 6 inch block	1,764	770	2,160	122
	4.3	do	1,764	770	3,720	211
4	3.7	4-inch hole in track	1,764	930	3,720	210
	2.8	do	1,764	960	2,140	121
	1.8	Smooth track	1,159	870	1,140	98
	2.1	do	1,159	890	1,220	105
	6.0	4 by 6 inch block	1,159	980	1,820	157
	7.1	do	1,159	900	1,320	114
5	7.1	4-inch hole in track	1,159	1,050	1,920	166
	7.6	do	1,159	1,100	2,140	185
	3.0	Smooth track	838	400	660	79
	5.5	do	838	380	640	76
	9.9	4 by 6 inch block	838	480	780	93
	10.0	do	838	410	760	91
6	4.7	4-inch hole in track	838	550	1,180	141
	3.5	do	838	470	820	98
GROUP 4						
2	3.3	Smooth track	2,580	2,500	3,500	136
	4.4	do	2,580	2,650	3,750	145
	3.2	4 by 6 inch block	2,580	3,150	8,600	334
	4.1	do	2,580	2,950	8,700	338
	7.4	2-inch hole in track	2,580	3,000	7,500	291
	7.4	do	2,580	2,850	6,350	246
3	8.5	Smooth track	1,540	1,100	1,750	114
	7.2	do	1,540	1,100	1,300	85
	5.5	4 by 6 inch block	1,540	950	3,550	296
	4.8	do	1,540	850	3,900	253
	5.7	4-inch hole in track	1,540	1,000	4,250	276
	4.2	do	1,540	900	2,750	178

CONCLUSIONS REGARDING IMPACT LOADS TRANSMITTED TO
CULVERTS

Viewing the results obtained in the impact determinations from the standpoint of the design of culverts, which is, of course the ultimate reason for conducting the tests, it may be stated for the range of conditions covered in these experiments that culverts should be designed to carry loads equal to the dead loads upon them from the embankment materials plus from 150 to 200 per cent of $C_i \frac{T}{7}$ (see pp. 113) to allow for the effect of moving concentrated traffic loads.

The values used for C_i should be those calculated by the formulæ given on page 113.

The actual impact blows reaching the culvert tops vary greatly with accidental conditions accompanying the exigencies of actual traffic, weather, and soil conditions. The above allowances are believed to safely provide for the impact effects which are reasonably expected to occur occasionally, at least for trucks running at speeds up to 10 miles an hour.

As with static superloads, the effects of moving superloads are negligible for heights of embankment exceeding 5 feet for culverts up to 3.5 feet in width and above moderate heights for wider culverts.

(Continued from page 112)

The treatment of the piles extended from 2 feet below mean low water to one foot from the end of the pile, the treated length varying from six to sixteen feet. The pile was always dry when painting was started and was allowed to dry thoroughly after each successive coat. The painting was done with ordinary ealimine brushes and a penetration of from one-eighth to one-sixteenth inch was obtained.

The cost of painting the piles, if so stored that they are accessible, is very small. It would be advisable to arrange the storage so that as few piles as possible have to be moved for painting, by so spacing them that they can be reached on all sides for treatment.

The length of time between the application of the seal coat and driving should be not less than seven days.

Cheat Haven Dam, Cheat Haven, Pa.—This treatment, reported by H. H. Haggard, of Sanderson & Porter, Engineers, New York, was applied to the upstream face of the dam and to the interior of penstocks and scroll cases. Approximately 7,000 square yards of the dam face was given two coats of water-gas tar at the rate of about 0.285 gallons per square yard, followed by one coat of coal tar at a cost for both applications of about 30 cents per square yard. The work was all submerged in December, 1925, and has not since been exposed for observation.

The penstocks and scroll cases received two coats of water-gas and two coats of coal tar. The two coats of coal tar proved to be too heavy and showed a tendency to run before water was admitted to the penstocks. After the plant had been in operation a few months an examination showed that the coating in the penstocks was still intact, but in the scrolls, where velocity of water was relatively high, the coal tar was stripped clean in patches exposing the concrete surface, stained dark brown by the absorbed water-gas tar.

Observations in the tunnels of the dam indicated that extremely little water had penetrated the concrete. Water had entered the tunnels at some con-

traction joints, but it was not expected that they would be tight. The penstocks and scrolls leaked somewhat in spots, which might have been avoided if the waterproofing could have been applied under more nearly perfect conditions.

In connection with the waterproofing of the dam, concrete test cylinders were made up of a 1:3:5½ mix of wet and dry consistency, cured in damp sand and air for 14 and 3 days, respectively, and given two coats of water-gas tar and one coat of coal tar. On completion of the treatment the test pieces were immersed in water for 7 days and the gain in weight after this period was found to be 1.4 per cent for the wet and 0.9 per cent for the dry-mixed concrete.

Abutment of Hancock-Sullivan Bridge over Taunton River, Me.—It is known that a treatment of four coats of water-gas tar and a seal coat of coal tar has been applied below water level; but no details of the treatment have been obtained.

GENERAL SUMMARY AND CONCLUSIONS

From the results of the tests both in the field and laboratory the following conclusions seem warranted.

1. That water-gas tar of the proper quality is readily absorbed by cement mortar and concrete, the rate of absorption varying with the manner of curing, age and density of the mix. Concrete of a 1:3:6 mix cured, respectively, 48 hours and 7 days under moist conditions in the forms followed by 7 days' exposure to dry air, was found to be the most absorptive, while a 1:1½:3 mix, cured 7 days in forms and 83 days in air, was the least absorptive.

2. That the absorption of coal tar by concrete is similar to that of water-gas tar except that the quantity absorbed increases with the time of exposure after treatment with water-gas tar.

3. That a treatment consisting of 4 coats of water-gas tar applied at the rate of about one-fourth gallon per square yard of surface, followed by one coat of coal tar appears to afford adequate protection against alkali attack, provided the concrete is of good quality, has been properly fabricated and not leaner than a 1:2:4 mix.

PARAFFIN SOLUTION USED ON MOLDS FOR CEMENT
MORTAR BRIQUETTES

Paraffin solution has been found to be much better than oil for preventing cement mortar from adhering to briquette molds, glass plates, or any other laboratory apparatus, according to R. B. Dayton, materials engineer of the State Road Commission of West Virginia.

A 6 to 7 per cent solution of paraffin in carbon tetrachloride is used instead of the customary heavy mineral oil. It may be applied with either a brush or rag. The surfaces of the molds are very easy to clean, requiring but slight brushing with a stiff fiber brush. The top and bottom of briquette molds require a slight scraping with a small trowel to remove the mortar, as the process of forming removes the paraffin coating, but this also happens when oil is used.

One advantage of the paraffin solution over oil is that the paraffin solution is quick drying and does not become incorporated with the mortar as sometimes happens with excess oil on the molds and plates.

EFFECT OF QUALITY OF PORTLAND CEMENT UPON THE STRENGTH OF CONCRETE

A REPORT ON TESTS CONDUCTED JOINTLY BY THE MICHIGAN STATE HIGHWAY DEPARTMENT AND THE BUREAU OF PUBLIC ROADS¹

Reported by F. H. JACKSON, Engineer of Tests, Bureau of Public Roads

During the last several years Portland cement concrete has been the subject of a great deal of study. Many tests have been made to determine the effect of such factors as quantity of water, quantity of cement, character and gradation of aggregates, etc., on the quality of the product. However, the effect of the quality of cement upon the quality of the concrete has received very little attention, the tacit assumption being, apparently, that any variations in the quality of the cement would be of relatively small importance, in so far as the quality of the resulting concrete is concerned, provided the cement passed the minimum requirements of the American Society for Testing Materials.

The results of tests made jointly by the Michigan State Highway Department and the Bureau of Public Roads at the Ann Arbor laboratory of the former during the summer of 1926 and presented in this report show that such an assumption may not hold under all conditions, and call attention to the desirability of studying those factors, including the cement, which affect the rate of hardening of paving concrete, because of the economic importance of this feature in influencing the time necessary to keep the completed pavement out of service.

Although the number of specimens subjected to tests was not large enough to justify general conclusions, their remarkable consistency points strongly to the probability that the strength of concrete in tension, flexure, and compression varies directly with the tensile strength of standard 1:3 mortar briquettes made of the cement and Ottawa sand. They indicate also that the strength of a concrete pavement at the end of the customary curing period may or may not be as great as the assumed strength, depending upon the character of the cement used, all other elements being the same.

It is now quite generally recognized that the tensile and flexural strength of concrete are of more significance than the crushing strength in determining the value of the product for use in pavement construction. Other things being equal, the distance apart at which transverse cracks will form in a plain concrete pavement will vary directly with the tensile strength of the concrete. Moreover, modern methods of design utilize the flexural strength of the concrete as a basis for calculating the thickness of pavement necessary to carry the maximum loads which will be allowed upon it. Unless traffic is restricted the maximum load is just as apt to come upon the pavement the day the road is opened as at some later period. Therefore, the critical strength of the concrete is the strength which it has attained at the time traffic is allowed upon the pavement. Any increase in strength which the concrete may attain subsequently should, in general, be considered only in the light of an additional factor of

safety and should not be utilized in design. This, of course, assumes that there will be no retrogression in strength at any later period. The rate of hardening, therefore, becomes an important item and should control the time necessary to keep the pavement closed to traffic.

It was for the purpose of determining the effect of variations in the quality of certain Portland cements used in the State of Michigan upon the strengths of paving concrete at the end of the curing period, as well as the relative rate of increase in strength up to and subsequent to this critical stage of the pavement's history, that the cooperative investigation was undertaken.

CHARACTER OF THE CEMENT THE ONLY VARIABLE IN THE TESTS

All concrete specimens used in the study were of the standard paving mix used by the Michigan State Highway Department, which is 1:2:3½ by volume. The only variable was the quality of the Portland cement, of which three grades were used—a slow-hardening grade with a briquette strength averaging about 200 pounds per square inch, a grade of medium strength running about 250 pounds per square inch, and a cement of fairly high early strength testing approximately 300 pounds per square inch, all at seven days.

A single consistency, corresponding to good average paving practice, was used throughout. Typical aggregates, a sand and gravel conforming in all respects to the requirements of the Michigan State highway specifications were employed; and all proportioning, mixing, molding, curing, and testing were strictly in accordance with the practice recommended by the American Society for Testing Materials. The results of routine tests on the three Portland cements used are given in Table 1, and the gradation and physical properties of the aggregates in Table 2. Thus far the specimens have been tested at the age of 7, 14, 21, 28, 90, and 180 days. Specimens for one and two year tests have been made and will be tested at the proper time.

TABLE 1.—Results of tests on three Portland cements used in Michigan concrete tests

Type of cement	Fine- ness— re- tained on No. 200 sieve	Setting time				Tensile strength	
		Initial		Final		7 days	28 days
		Hrs	Min.	Hrs.	Min.	Lbs. per sq. in.	Lbs. per sq. in.
High strength.....	Per cent 18.5	4	10	5	40	325	400
Medium strength.....	18.7	3	55	5	55	255	380
Low strength.....	20.0	4	10	6	00	180	315

Three types of tests have been made—direct tension, flexure, and compression. For the tension tests, cylinders 6 inches in diameter by 21 inches long were cast and tested in accordance with the method first

¹ These tests were conducted at the Ann Arbor laboratory of the Michigan State Highway Department, under the joint supervision of R. L. Morrison, director of the laboratory, and C. E. Froudeley, formerly assistant engineer of tests, Bureau of Public Roads.

TABLE 2.—Results of tests of aggregates used in Michigan concrete tests

SAND			GRAVEL		
Size:			Size:		
Retained on No. 10 sieve	per cent.	28	Retained on 1½-inch screen	per cent.	0
Retained on No. 30 sieve	per cent.	75	Retained on 1-inch screen	per cent.	30
Retained on No. 50 sieve	per cent.	93	Retained on ¾-inch screen	per cent.	60
Retained on No. 100 sieve	per cent.	98	Retained on ½-inch screen	per cent.	85
Color	O. K.		Retained on ¼-inch screen	per cent.	100
Weight per cubic foot	pounds	112	Per cent of wear (gravel test)	per cent.	5
Strength ratio, 7 days		138	Weight per cubic foot	pounds	109
Strength ratio, 28 days		135			

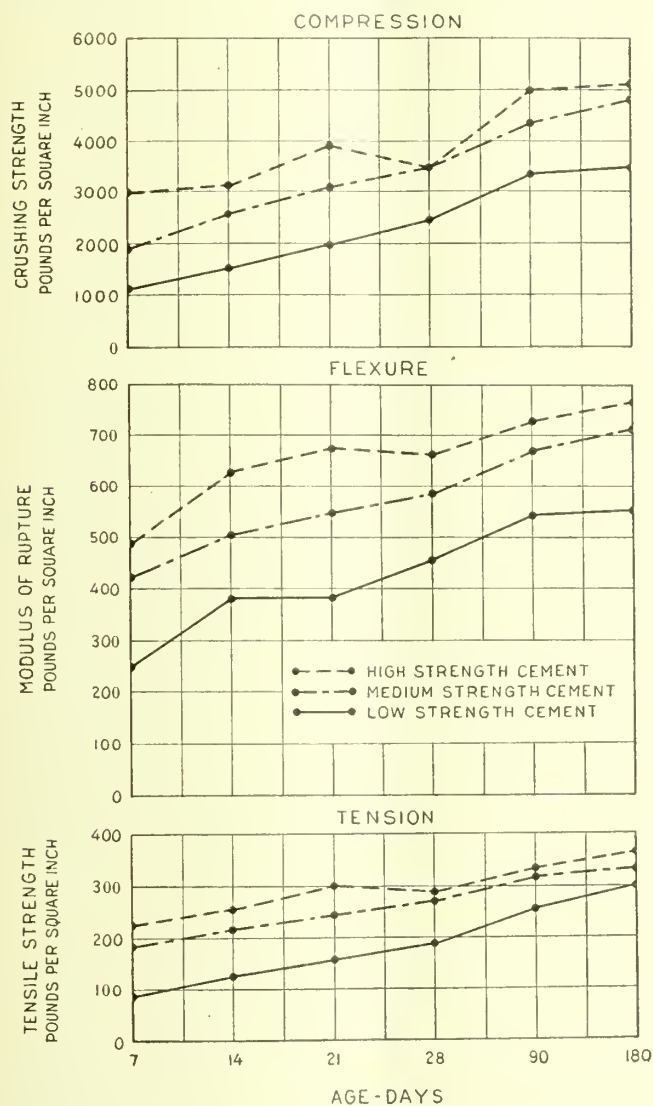


FIG. 1.—STRENGTH OF CONCRETE MADE FROM THREE GRADES OF PORTLAND CEMENT

used in the structural materials research laboratory at Lewis Institute, Chicago (now the research laboratory of the Portland Cement Association), and described in the Proceedings of the American Society for Testing Materials for 1926.² By this method the cylinders are broken in a universal testing machine. The grips consist of two pieces of 6-inch steel pipe lined with leather and split part way along four elements so as to slip over the ends of a cylindrical concrete specimen. The segments are then drawn tight by means of tangential bolts to prevent slipping during

the application of the load. Ball and socket joints in each grip make the set-up self-aligning. In general, two breaks were secured from each specimen.

The specimens for the flexure tests were beams 6 by 6 by 36 inches in size, tested as cantilevers by the method first proposed by Clemmer.³ The beams are supported as cantilevers and an extension arm with a container at the extreme end placed over the free end of the beam. Load is applied by an even flow of shot or water from a separate container equipped with a quick-acting valve. Calculations to determine the flexural strength involve the overhang of the specimen, the extension arm, and the weight of shot or water required to cause failure. Two breaks were secured from each beam.

The compressive tests were made in the conventional manner on 6 by 12 inch cylinders; and in these, as in the other tests, the specimens were tested immediately on removal from moist storage.

The schedule of tests called for three specimens of each type for each age and for each cement. As it was possible in almost all cases to obtain two breaks from each of the tension and flexure specimens, six values for tensile and flexural strength were obtained and reported for each combination and three values in compression. The results of all tests up to and including the six-month period are given in Tables 3 to 5, inclusive. The average strengths shown in these tables for each of the three cements tested are plotted against age in Figure 1, and in Figure 2 the results of the three types of concrete tests at the age of 7 and 28 days are plotted against the tensile strengths of the cements as shown by 1:3 Ottawa sand briquette tests at the same periods.

TABLE 3.—Results of tests of 1:2:3½ gravel concrete specimens made with high-strength cement

Type of test	Age in days					
	7	14	21	28	90	180
Compressive strength in pounds per square inch of 6 by 12 inch cylinders.	3,073	3,400	3,840	3,060	4,900	5,115
$\frac{W}{C} = 0.76$.	2,890	2,865	3,630	3,995	5,200	5,100
	2,970	3,168	4,220	3,310	4,810	5,020
Average	2,978	3,144	3,897	3,455	4,970	5,078
Tensile strength in pounds per square inch of 6 by 21 inch cylinders.	203	232	297	276	334	371
$\frac{W}{C} = 0.78$.	224	262	312	303	316	374
	237	279	300	294	337	363
	235	246	282	274	352	352
		273	310	263	361	
Average	225	258	299	282	336	365
Modulus of rupture in pounds per square inch of 6 by 6 by 36 inch beams, tested as cantilevers.	483	678	542	625	702	781
$\frac{W}{C} = 0.75$.	465	715	615	668	708	780
	532	664	818	678	745	729
	498	654	858	703	790	684
	462	495	672	665	706	834
	497	562	548	620	682	786
Average	489	628	676	660	722	766

BRIQUETTE STRENGTH OF CEMENT AND STRENGTH OF CONCRETE RELATED

The effect of variations in the quality of cement upon the strength of the concrete, as shown by Figure 1, appears to be quite marked. Observing the relative values for each of the three grades at the end of the conventional curing period of 21 days, it will be seen that the strength of the concrete containing the high-test cement is approximately double that of the concrete

² Proc. A. S. T. M., 1926, pt. 2, p. 447.³ Proc. A. C. I., 1926, p. 304.

containing low-test cement, and that the concrete in which the medium-strength cement was used runs about midway between. The significance of such wide variations in strength from the design standpoint will be readily apparent.

TABLE 4.—Results of tests of 1 : 2 : 3½ gravel concrete specimens made with medium-strength cement

Type of test	Age in days					
	7	14	21	28	90	180
Compressive strength in pounds per square inch of 6 by 12 inch cylinders.	1,881	2,670	3,435	3,383	4,320	4,762
W	1,816	2,540	2,880	3,610	4,390	4,895
$C = 0.76$	2,000	2,535	2,992	3,500	4,200	4,685
Average.....	1,899	2,552	3,102	3,497	4,303	4,781
Tensile strength in pounds per square inch of 6 by 21 inch cylinders.	194	273	297	309	344	
W	207	259	299	303		
$C = 0.78$	225	215	291	315		
	211	246	346			
	178	222	273	329	332	
	177	210	235	333		
Average.....	183	215	242	270	319	330
Modulus of rupture in pounds per square inch of 6 by 6 by 36-inch beams tested as cantilevers.	456	525	487	576	694	743
W	430	500	531	554	686	749
$C = 0.75$	447	544	627	592	666	669
	442	469	568	625	703	664
	376	487	551	591	619	702
	393	495	512	588	639	716
Average.....	424	503	549	588	668	707

TABLE 5.—Results of tests of 1 : 2 : 3½ gravel concrete specimens made with low-strength cement

Type of test	Age in days					
	7	14	21	28	90	180
Compressive strength in pounds per square inch of 6 by 12 inch cylinders.	1,051	1,230	2,000	2,482	3,260	4,055
W	1,158	1,760		2,290	3,412	3,195
$C = 0.76$	1,150	1,710	1,970	2,470	3,250	3,250
Average.....	1,120	1,567	1,985	2,414	3,307	3,500
Tensile strength in pounds per square inch of 6 by 21 inch cylinders.		112	143	153	240	305
W	86		174	210	285	
$C = 0.78$		124	155	196	253	314
	89	145	166	196		309
	88		162	189	262	275
			150	174	225	
Average.....	88	127	158	187	253	301
Modulus of rupture in pounds per square inch of 6 by 6 by 36 inch beams tested as cantilevers.		442	337	428	570	558
W	255	450	341	524	599	587
$C = 0.75$	222	332	435	423	519	529
	244	359	449	456	571	537
	252	377	361	486	486	585
	268	345	372	447	410	520
Average.....	248	384	383	461	513	553

If it be assumed that the edge thickness of a concrete pavement has been determined by the corner formula on the assumption that the concrete will have a modulus of rupture of 600 pounds per square inch at the time the road is opened to traffic, it will be found from Figure 1 that, with the aggregates employed and for the conditions obtaining in these tests, the required modulus of rupture was attained in approximately 12 days by the use of the high-strength cement, in some-

what more than 28 days when the medium-strength cement was employed, and that it had not been reached up to six months when the low-strength cement was used. The low-strength cement did not pass the requirements for strength of the American Society for Testing Materials, but if, for this reason, we ignore the results of the tests on the concrete in which it was used, we still find the time required to produce concrete of the required quality to be markedly longer when the medium-strength cement (which passed all A. S. T. M. requirements) was used than when the high-strength cement was employed.

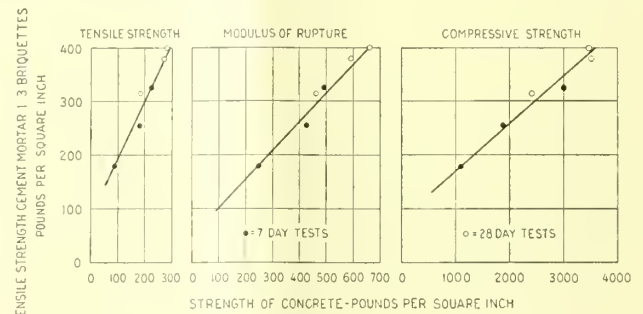


FIG. 2.—RELATION BETWEEN QUALITY OF CEMENT AS SHOWN BY ROUTINE TENSILE TESTS OF BRIQUETTES AND THE QUALITY OF CONCRETE (1 : 2 : 3½ BY VOLUME) FOR THREE GRADES OF PORTLAND CEMENT

Referring now to Figure 2, it will be observed that there is a very interesting relation between the results of briquette tests on the three cements at 7 and 28 days and the results of the various concrete tests at the same periods. There appears to be a very definite relation between briquette strength and concrete strength at corresponding ages. This relation, moreover, applies to all three types of concrete tests. It will not be asserted, of course, that these results are sufficient to warrant any conclusions of a general nature. It is believed, however, that the meagerness of the data presented is to a certain extent balanced by the remarkable concordance of the results obtained, so that they may be said to be at least indicative and, therefore, of value in suggesting a field for additional and more extensive research.

REPORT AVAILABLE ON RUN-OFF FROM SMALL AGRICULTURAL AREAS

An article entitled "Run-off from small agricultural areas," by C. E. Ramser, drainage engineer, reprinted from the Journal of Agricultural Research, volume 34, No. 9, will be of interest to highway engineers. The report deals with rainfall and run-off measurements made on six watersheds ranging in area from 1¼ to 112 acres in Madison County, Tenn. The results are applicable to the design of all types of drainage structures where similar conditions exist. Copies of this report may be obtained without charge from the Office of Information, United States Department of Agriculture.

CAPPING SQUARE FOR CONCRETE COMPRESSIVE STRENGTH SPECIMENS

By F. V. REAGEL, Engineer of Materials, Missouri State Highway Department

The road materials testing laboratory of the Missouri State Highway Department has for some time been using a device for capping concrete cylinders for compression tests which gives the specimens a uniform bearing and at the same time permits the work to be

structuring the framework to prevent warping, and the inside faces are lined with 16-gauge metal. The slot shown just above the base is to receive a square metal plate, from three-eighths to one-half inch in thickness, on which the specimens are capped. For convenience in handling, three rivet or bolt heads are inserted in the bottom of the plate.

CYLINDERS CAPPED ACCURATELY AND RAPIDLY

The plate upon which the specimen is to be capped is first given the usual coating of oil to prevent sticking. Sufficient capping material for one cap plus a slight excess is then deposited upon the plate. The plate is shoved into the corner snugly, the groove at the bottom allowing the specimen to rest against the corner and still remain near the center of the plate. The specimen is lowered upon the capping material with a rotating motion under considerable pressure, the motion finally ending with the specimen resting snugly against each side of the corner of the capping square. The rotating motion serves to squeeze out the excess capping material which is rapidly worked up around the outside of the specimen. A large trowel is then inserted under the plate and the plate and specimen set aside until the cap has hardened.

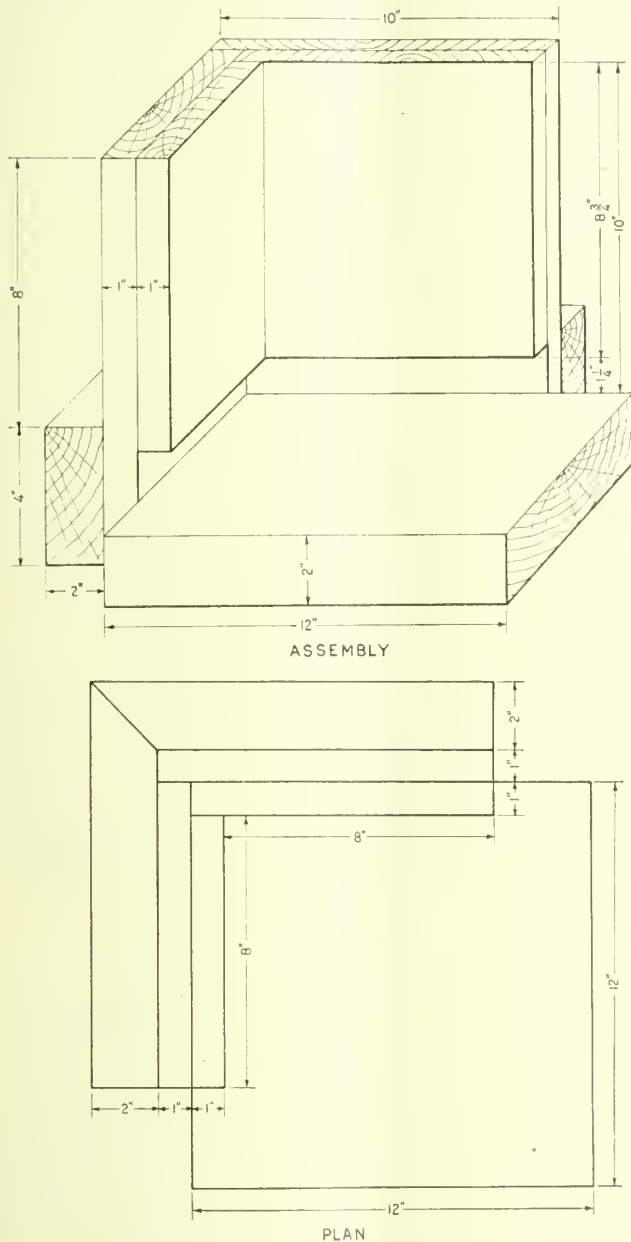


FIG. 1.—SHOWING DETAILS OF CONSTRUCTION OF CAPPING SQUARE

done rapidly. This device was developed by A. D. Conrow, formerly assistant testing engineer and now employed in the same capacity by the Kansas Department of Highways. Comments of visiting engineers have indicated that the method which has been developed will be of general interest to testing engineers.

Figure 1 shows the details of design of the apparatus, and with this guide it can be constructed at small expense. Well-seasoned wood should be used for con-



CAPPING SQUARE IN USE AND CAPPED SPECIMENS

As soon as the cap has sufficiently hardened the plate is loosened by tapping lightly against the edge, and the process is repeated with the other end of the specimen upon the plate. Using this method, one operator can cap about 80 specimens on each end per day and the resulting specimens will have uniformly parallel ends.

The capping materials in use at present are a mixture of plaster of Paris and Portland cement for hand-cast specimens and a 1 to 3 mortar of Lumnite cement and sand for the more irregular field-drilled specimens.

The illustration shows the capping device in use. On account of the extreme irregularity of some specimens drilled from pavements, this process is not always suitable for use in capping each end of the specimen. In such cases we use the metal collar shown on the specimen in the extreme right of the picture. This collar is of 18-gage metal and is provided with two wing-nut bolts by means of which the collar may be drawn tightly around the specimen. Capping material is then placed within the collar, building up the entire specimen to the height of the longest element plus the desired thickness of cap.

NEW RESEARCH PROJECTS INITIATED BY BUREAU OF PUBLIC ROADS

Six new highway research projects have been initiated by the United States Bureau of Public Roads during the current year. Announcement has been in previous issues of PUBLIC ROADS of the Yadkin River bridge tests and of tests of anchorage and bond of pile heads in concrete. The more recent projects include investigations of motor-truck impact as influenced by road type, motor-truck impact as influenced by wheel type, statistical analysis of highway grade crossing accidents in 1926, and the effect of type and gradation of coarse aggregate upon the flexural strength of plain concrete.

MOTOR-TRUCK IMPACT INVESTIGATIONS

The investigations of motor-truck impact as influenced by road type and as influenced by wheel type constitute two separate but related investigations. The first of these is for the purpose of determining the cushioning properties of the various road types under controlled conditions of motor-truck impact. A number of road types such as plain concrete, sheet asphalt on concrete base, bituminous concrete on concrete base, and penetration macadam will be selected in the vicinity of Washington, and on these sections motor-truck impacts will be produced identical as to conditions of drop, load, and tire equipment. Comparison between pavement types will then be made on the basis of the magnitude of the impact developed on each under exactly comparable test conditions. Any cushioning properties inherent in the pavement types should tend to decrease the magnitude of the impact.

Two methods are possible for producing the impact forces. The impact machine which has been in use for some years on the slab impact tests can be made portable. This machine is designed to give a wide range of impact blows, and measurement would be made of the height of drop of a wheel representing various conditions of truck load and equipment and the impact reaction. It is also planned to have a truck used in other impact investigations pass over artificial obstructions and to compute the impact forces produced from the record of an accelerometer attached to the truck. The first tests will probably be made under conditions representing a worn tire and heavy truck; and if the differences in impact reactions warrant, the program will be amplified to include other conditions. The tests on bituminous types will include as wide a range as possible of temperature conditions.

The investigation to determine the relative cushioning effect of various types of motor-truck wheels will be similar to but less extensive than the recently completed tests with various tire equipments. Wheels to be tested will include steel disk, wood spoke, and cushion types.

A 2-ton truck will be loaded so as to have 2,500 pounds on each rear wheel and later 5,000 pounds on each rear wheel, and equipped with dual pneumatic, dual new cushion, and dual worn solid tires, and will be

operated at various speeds over different types of pavement and artificial obstacles. Impact forces will be measured by the coil-spring accelerometer and comparisons made where all conditions are constant except the type of wheel.

GRADE-CROSSING ACCIDENTS TO BE STUDIED

A statistical analysis is to be made of highway-railroad grade-crossing accidents in 1926, as reported by the Class I steam railroads to the Interstate Commerce Commission. The object is to determine the relative frequency of grade-crossing accidents in rural and urban areas. The data may be used later in an attempt to establish the correct ratios which grade-crossing accidents in the rural areas bear to the total of all the highway accidents in those areas. It is also desired to discover any other significant evidence as to causes and conditions of accidents which may be revealed in a mass analysis of the 5,890 accidents reported in 1926.

The Bureau of Statistics of the Interstate Commerce Commission has on file a complete set of individual accident reports from all railroads under its jurisdiction. Pertinent data will be transcribed from those involving highway-railroad grade crossings and analyzed by means of tabulating machines.

EFFECT OF GRADING AND TYPE OF CONCRETE AGGREGATE TO BE STUDIED

This project is to determine how quality and economy in concrete construction are effected by variations in type and grading of coarse aggregate. It is planned to obtain a number of representative coarse aggregates such as trap, granite, limestone, sandstone, and dolomite rock, and glacial, siliceous, and calcareous gravels, and blast-furnace slag, selecting a total of 18 aggregates in all. These are to be prepared in six different gradings, as follows:

Coarse aggregate gradations

Grading No.	Total passing, square opening				
	1/8-inch	1/4-inch	3/4-inch	1 1/4-inch	2-inch
	Per cent	Per cent	Per cent	Per cent	Per cent
1.	0	0	15	40	100
2.	0	0	30	55	100
3.	0	5	45	70	100
4.	0	5	45	100	100
5.	0	10	65	100	100
6.	0	10	100	100	100

Flexure tests are to be made on cantilever beams made of concrete with the above coarse aggregate types and gradings as the only variables. Four mixes are to be used ranging from 1:1 1/2:3 to 1:2:4, making a total of 432 combinations.

Accurate measurements of yield will be made on all combinations. Compression tests will also be made, using the fractured prisms resulting from the flexure tests. All tests will be made at 28 days.

ROAD PUBLICATIONS OF BUREAU OF PUBLIC ROADS

Applicants are urgently requested to ask only for those publications in which they are particularly interested. The Department can not undertake to supply complete sets nor to send free more than one copy of any publication to any one person. The editions of some of the publications are necessarily limited, and when the Department's free supply is exhausted and no funds are available for procuring additional copies, applicants are referred to the Superintendent of Documents, Government Printing Office, this city, who has them for sale at a nominal price, under the law of January 12, 1895. Those publications in this list, the Department supply of which is exhausted, can only be secured by purchase from the Superintendent of Documents, who is not authorized to furnish publications free.

ANNUAL REPORTS

Report of the Chief of the Bureau of Public Roads, 1924.
Report of the Chief of the Bureau of Public Roads, 1925.

DEPARTMENT BULLETINS

- No. 105D. Progress Report of Experiments in Dust Prevention and Road Preservation, 1913.
- *136D. Highway Bonds. 20c.
- 220D. Road Models.
- 257D. Progress Report of Experiments in Dust Prevention and Road Preservation, 1914.
- *314D. Methods for the Examination of Bituminous Road Materials. 10c.
- *347D. Methods for the Determination of the Physical Properties of Road-Building Rock. 10c.
- *370D. The Results of Physical Tests of Road-Building Rock. 15c.
- 386D. Public Road Mileage and Revenues in the Middle Atlantic States, 1914.
- 387D. Public Road Mileage and Revenues in the Southern States, 1914.
- 388D. Public Road Mileage and Revenues in the New England States, 1914.
- 390D. Public Road Mileage and Revenues in the United States, 1914. A Summary.
- 407D. Progress Reports of Experiments in Dust Prevention and Road Preservation, 1915.
- *463D. Earth, Sand-Clay, and Gravel Roads. 15c.
- *532D. The Expansion and Contraction of Concrete and Concrete Roads. 10c.
- *537D. The Results of Physical Tests of Road-Building Rock in 1916, Including all Compression Tests, 5c.
- *583D. Reports on Experimental Convict Road Camp, Fulton County, Ga. 25c.
- *660D. Highway Cost Keeping. 10c.
- *670D. The Results of Physical Tests of Road-Building Rock in 1916 and 1917. 5c.
- *691D. Typical Specifications for Bituminous Road Materials. 10c.
- *724D. Drainage Methods and Foundations for County Roads. 20c.
- *1077D. Portland Cement Concrete Roads. 15c.
- *1132D. The Results of Physical Tests of Road-Building Rock from 1916 to 1921, Inclusive. 10c.

DEPARTMENT BULLETINS—Continued

- No. 1259D. Standard Specifications for Steel Highway Bridges, adopted by the American Association of State Highway Officials and approved by the Secretary of Agriculture for use in connection with Federal-aid road work.
- 1279D. Rural Highway Mileage, Income, and Expenditures, 1921 and 1922.
- 1486D. Highway Bridge Location.

DEPARTMENT CIRCULARS

- No. 94C. TNT as a Blasting Explosive.
- 331C. Standard Specifications for Corrugated Metal Pipe Culverts.

MISCELLANEOUS CIRCULARS

- No. 62M. Standards Governing Plans, Specifications, Contract Forms, and Estimates for Federal Aid Highway Projects.
- 93M. Direct Production Costs of Broken Stone.
- 105M. Federal Legislation Providing for Federal Aid in Highway Construction and the Construction of National Forest Roads and Trails.

FARMERS' BULLETINS

- No. *338F. Macadam Roads. 5c.
- *505F. Benefits of Improved Roads. 5c.

SEPARATE REPRINTS FROM THE YEARBOOK

- No. *739Y. Federal Aid to Highways, 1917. 5c,
- *849Y. Roads. 5c.
- 914Y. Highways and Highway Transportation.

REPRINTS FROM THE JOURNAL OF AGRICULTURAL RESEARCH

- Vol. 5, No. 17, D- 2. Effect of Controllable Variables upon the Penetration Test for Asphalts and Asphalt Cements.
- Vol. 5, No. 19, D- 3. Relation Between Properties of Hardness and Toughness of Road-Building Rock.
- Vol. 5, No. 24, D- 6. A New Penetration Needle for Use in Testing Bituminous Materials.
- Vol. 6, No. 6, D- 8. Tests of Three Large-Sized Reinforced-Concrete Slabs Under Concentrated Loading.
- Vol. 10, No. 5, D-12. Influence of Grading on the Value of Fine Aggregate Used in Portland Cement Concrete Road Construction.
- Vol. 11, No. 10, D-15. Tests of a Large-Sized Reinforced-Concrete Slab Subjected to Eccentric Concentrated Loads.

* Department supply exhausted.

STATUS OF FEDERAL AID HIGHWAY CONSTRUCTION

JULY 31, 1927

* Includes projects reported completed (final vouchers not yet paid) totaling: Estimated cost \$ 88,969,279.12 Federal aid \$ 39,575,452.01 Miles 3,550.6

PUBLIC ROADS

A JOURNAL OF HIGHWAY RESEARCH



UNITED STATES DEPARTMENT OF AGRICULTURE
BUREAU OF PUBLIC ROADS



VOL. 8, NO. 7



SEPTEMBER, 1927



A CALIFORNIA HIGHWAY TREATED WITH LIGHT ASPHALTIC OIL

PUBLIC ROADS

A JOURNAL OF HIGHWAY RESEARCH

U. S. DEPARTMENT OF AGRICULTURE

BUREAU OF PUBLIC ROADS

CERTIFICATE: By direction of the Secretary of Agriculture, the matter contained herein is published as administrative information and is required for the proper transaction of the public business

The reports of research published in this magazine are necessarily qualified by the conditions of the tests from which the data are obtained. Whenever it is deemed possible to do so, generalizations are drawn from the results of the tests; and, unless this is done the conclusions formulated must be considered as specifically pertinent only to the described conditions

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R. E. ROYALL, Editor

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Owing to the necessarily limited edition of this publication it will be impossible to distribute it free to any persons or institutions other than State and county officials actually engaged in planning or constructing public highways, instructors in highway engineering, periodicals upon an exchange basis, and Members of both Houses of Congress. Others desiring to obtain "Public Roads" can do so by sending 10 cents for a single number or \$1 per year to the Superintendent of Documents, U. S. Government Printing Office, Washington, D. C.

LIGHT ASPHALTIC OIL ROAD SURFACES

Reported by C. L. McKESSON, Materials and Research Engineer, California Department of Public Works, Division of Highways, and W. N. FRICKSTAD, Highway Engineer, United States Bureau of Public Roads¹

IN ALL States the problem of developing satisfactory types of surface for light-traffic roads is an important one because of the large mileage of such roads in the county and local systems, and even in the systems that are set apart for improvement by the States.

In the Western States the need for such types of construction is felt, perhaps more sharply than in other sections, not only because of the somewhat more extensive mileage on which traffic is now and will remain light, but also because similar forms of construction are needed for the initial improvement of a very considerable mileage upon which a rapid increase of traffic is expected. On roads of the latter class the inexpensive surface is regarded as merely a first stage of improvement, to be replaced as traffic demands and funds become available; but for the present the problem is one of improving the service of such roads at minimum expense in order that the benefit may be quickly and widely distributed over a large mileage.

To meet this need the States of the West have built in recent years many miles of traffic-bound metaled roads, the surfacing material of which is finely crushed rock or gravel. Differing from waterbound macadam in several aspects this type has practically supplanted true macadam because it is less expensive in first cost and more readily maintained under traffic. However, such surfaces wear rapidly under traffic, and highway authorities are becoming seriously concerned at the mounting cost of renewing the road metal. Also, the dust nuisance is very annoying, costly and even dangerous, and the cost of operating vehicles upon these surfaces is rather high.

Recognizing the objectionable features of present practices, and convinced of the necessity of developing satisfactory types of highway surfacing intermediate between ordinary gravel and sand-clay and the expensive pavement types, and having in mind, especially, the need of such types in the Western States, the United States Bureau of Public Roads and the California Highway Commission, in 1926, undertook the cooperative investigation of which the preliminary findings are presented in this report.

The particular purposes of the investigation were to determine the service value that may be expected of fine crushed rock and gravel surfaces; to ascertain what methods might be employed to conserve material and increase serviceability by the use of bituminous material; and to study their value as bases for higher types of less cost than the plant-mixed surfaces. Rather than undertake experimental construction, it was hoped the solution would be found in an examination and analysis of surfaces already existing.

The first effort upon inauguration of the study was the accumulation of data regarding successful and unsuccessful examples of the respective types, records of processes and materials, cost data, weather and soil information, service history, and maintenance methods and costs. These examples were found to fall within three natural groups:

1. Untreated surface, generally of the fine crushed rock or gravel type.

2. Roadway, either natural soil or metaled, treated with "light" or "fuel" asphaltic oil, material which can be applied at atmospheric temperature or by gentle warming.

3. Metaled roadways treated with "road" oil or with soft grades of asphalt.

The multiplicity of roadway types within the field of the study even when limited to the Western States, has made clear that much time will be required to complete the work undertaken. In the meantime, the disadvantages of untreated surfaces have become all too apparent and everywhere there is a desire to find the remedy at the earliest possible date. Many experiments are being undertaken, some of which repeat methods discarded elsewhere. Therefore a progress report has been prepared for publication by the California Division of Highways, and the essential portions of this report are reproduced in this article. This is done to make immediately available to highway authorities certain information that has been gathered as to the use of "light" asphaltic oil residuum in the two Pacific coast States of Oregon and California. Field, office, and laboratory data pertaining to the second group of examples above mentioned are presented and discussed; and sufficient data pertaining to the first group, "Untreated surfaces," are included to tie the main subject to the present construction and maintenance situation.

Although "road" oils and soft asphalts overlap the field of "fuel" oil, and some mention thereof will necessarily find its way into this progress report, the use of the former has not been examined sufficiently to warrant presentation at this time. The study of surface treatments with the heavier asphaltic products, and the possibility of utilizing existing crushed rock and gravel surfaces as bases for bituminous macadam construction, are the purposes of the extension of the investigation. Consequently, no general conclusions as to relative utility of types are yet formulated. Such conclusions as are here presented relate to the scope of this progress report only.

FINE CRUSHED ROCK AND GRAVEL SURFACES

As commonly constructed, the fine crushed rock and gravel surfaces are built in two courses to a total thickness of about 6 inches. The maximum size of the base stone is approximately $1\frac{1}{2}$ inches and that of the top course, 1 inch or $\frac{3}{4}$ inch. Crusher-run material is used for both courses—often with an admixture of clay or other binder—and both courses are compacted under traffic, the construction trucking being utilized as much as possible for this purpose.

Such surfaces are maintained by frequent blading or dragging or by a combination of the two. Ideally, there should be maintained on the surface of the road a light mulch, consisting of about half an inch of loose fine material to protect the compacted metal beneath; and the constant working of this fine material across the surface has been found effective as a means of preventing the formation of pits and corrugations.

¹ B. A. Anderton, formerly of the Bureau of Public Roads, collaborated in the field investigations on which this report is based.

Intended for light traffic, and serving best where the traffic does not exceed 300 vehicles per day, these roads can usually be kept smooth at a yearly cost of about \$1 per mile for each unit of average daily traffic. For example, a road carrying an average traffic of 300 vehicles per day can be kept smooth at a cost of about \$300 per mile per year.

This, however, does not include the cost of replacing the material worn from the surface by the traffic, a loss which is very considerable, and which, with the accompanying dust nuisance, is perhaps the most serious objection to the types. Estimating from the interval between resurfacing operations and the amount of metal added, reports received indicate that from 1 to 1½ inches of metal will be removed in a year by a traffic of 500 vehicles a day. It is possible that these are extreme cases, and that the heavy loss indicated is the result of imperfect compacting or the use of inferior material; but even if the estimates be heavily discounted, and the loss be assumed to average no more than three-fourths inch of compacted material a year, the annual cost of replacement, at \$2.50 per cubic yard, loose measurement, will be \$750 a mile of 18-foot surface. Adding to this the \$500 per mile required for ordinary maintenance, the total annual cost of upkeep for an 18-foot roadway, carrying 500 vehicles a day, is found to be no less than \$1,250 a mile; and this is a conservative figure, as indicated by the detailed cost data to be found elsewhere in this report.

The other objections that have been raised against the type are, (1) that the tractive resistance of the surface is high, and (2) that it causes heavy tire wear.

Recognizing these defects, the Oregon State Highway Commission, in 1923, conducted experiments with a view to developing methods of treating the fine crushed metal with bituminous materials to improve its service and reduce the heavy loss by attrition; and the successful use in these experiments of a light asphaltic residual oil, known locally as "fuel oil" led to the use of the material for the treatment of several hundred miles by the commission in 1924, 1925, and 1926.

Impressed by the Oregon experience, the California Division of Highways conducted similar experiments in 1925, which were followed by the oiling of 190 miles of State highway in 1926, and by the scheduling of about 700 miles for oiling in 1927; and the Washington and Idaho State Highway Departments also treated experimental sections in 1926, using methods adapted from Oregon practice.

OIL TREATMENT OF FINE CRUSHED ROCK AND GRAVEL ROADS

Two general methods of treatment have been developed, which may be described, respectively, as the method of surface treatment, and the method of surface mixing. These are briefly described as follows:

Surface treatment method.—This treatment as practiced in Oregon and California contemplates the impregnation of the upper portion of a compacted fine crushed rock or gravel road with light asphaltic oil. The process resembles the ordinary surface treatment of macadam when light oils or tars are used, but differs from surface treatments with heavy oils in that there is no formation of a distinct mat of stone, chips, and binder. For practical reasons, some cover material is commonly used, but the amount is usually limited, and the light oil so penetrates the road crust that there is finally little left on the surface to be absorbed by the application of chips.

The first step in this process is the thorough sweeping of the existing road surface with a power broom, supplemented by hand brooming if necessary, to remove all loose material and scales of fines and expose the compacted rock surface. The light oil is then applied under pressure at the rate of about three-tenths of a gallon per square yard and allowed to penetrate, the time required depending upon the texture of the surface and the viscosity of the oil. Usually two or three days is sufficient, during which time the road is closed to traffic if practicable. At this stage in Oregon small imperfections in the surface are repaired with a lean mixture of oil and aggregate.

When the first application of oil has thoroughly penetrated, a second is spread at the rate of about two-tenths of a gallon per square yard, and, as before, is allowed to stand for several days without traffic if possible, while the oil is being partially absorbed. The treatment is completed by spreading stone chips over the surface before the road is opened to traffic, and, if vehicles can not longer be kept off, the chips are spread immediately after the second application of oil. The quantity of cover material usually used ranges from 10 to 25 pounds per square yard depending upon the viscosity of the oil and whether or not the road is to be opened to traffic immediately. The larger amount is necessary if the road is to be opened immediately; and oils of high viscosity require more cover than the thinner oils.

There is a tendency in some places to modify the surface treatment method described by substituting heavy road oil. As before stated, no recommendations have yet been formulated on this subject. It should be pointed out, however, that heavy road oil is more costly than "fuel oil," demands more equipment and better technic in application, and requires from 100 to 200 cubic yards per mile of screenings for cover.

Surface mixing method.—In the method of surface mixing, the surface of the previously constructed fine crushed rock or gravel road is first scarified to a uniform depth of 1½ to 3 inches according to the thickness of bituminous surface desired. The light asphaltic residual oil is then applied in two or three applications, each consisting usually of about one-half gallon per square yard; and after each application the oil and loose stone are partially mixed with a disk cultivator or spring-tooth harrow, or both. The material is then bladed repeatedly into windrows and respread until a uniform color is attained. It is finally spread to the desired crown and the road is then opened and continually dragged or bladed while it is being compacted by the traffic.

Roads treated by these methods are of distinctive type and are not to be confused with roads treated for dust palliative purposes. In appearance they resemble other asphaltic surfaces. Skillfully prepared and treated, their smoothness shortly after completion, as indicated by "roughometer" readings, is equal to that of the best pavements. When well maintained, they appear to rate with good bituminous macadam; but, at their worst, when maintenance is neglected or when the surfacing material or its condition at the time of treatment is poor, they may become intolerably rough. There is no dust, and tractive resistance and tire wear are both low.

In the progress report which follows there are descriptions of what appear to be the best methods of constructing and maintaining fine crushed rock and gravel surfaces and of treating them with light asphaltic

residual oils according to the two general processes that have been developed. Comparative initial and annual cost data are presented to show the relative economy of treated and untreated roads; and observations of the character and condition of a number of roads are reported to show the effects of traffic, moisture and climatic conditions, quality of materials and their availability, upon the cost and serviceability of the roads, as a guide to the selection of one type or another under various conditions.

CONCLUSIONS

Although the investigation so far has been limited territorially and in subject matter as already stated, progress has been sufficient to justify a number of conclusions, particularly with reference to the economy of the light oil treatments for fine crushed rock and gravel roads, and the essential features of the construction methods. These, which are set forth below, are based principally upon data gathered in Oregon and California, but the climate, topography, soil, traffic, and costs vary so widely in the two States that the observations have a rather wide application.

The principal conclusions are as follows:

1. Light asphaltic residual treatments as practiced in Oregon and California have demonstrated their utility in the preservation of existing metaled road surfaces, and are justified in places where the cost of the oil is not prohibitive.

2. Oil treatment has substantially reduced maintenance and replacement charges on fine crushed rock and gravel roads in Oregon and California. The first cost is frequently less than the value of the metal lost yearly from the same road before treatment. Maintenance after oiling will probably not exceed the cost of adequate maintenance of the untreated surface, exclusive of the replacement of metal.

3. Oil treatment improves service and is, therefore, popular with the public. It eliminates dust, increases smoothness, decreases tire wear, and lowers fuel consumption.

4. Any road in Oregon or California which has justified improvement with a fine crushed rock or gravel surface warrants the additional expense of a light oil treatment, except possibly in rare instances of isolation, extraordinarily cheap material supply, or low service requirements. Direct financial justification will generally follow a comparison of the estimated maintenance after treatment plus a reasonable annual portion of the oiling cost with the known maintenance cost prior to oiling plus the value of untreated metal destroyed annually.

5. An efficient maintenance organization is essential to the success of light oil treatments. Maintenance must be immediate and continuous, otherwise oiling will be unsuccessful and should not be undertaken. While proper maintenance is stressed as a prime requisite, the total annual cost is less than that of untreated roads.

6. The recorded success with light oils should not stop continued development of present practices or experimentation with heavy asphalts. However, the most that can be expected of the latter is some further reduction of annual costs, particularly where traffic is heavy, and that possibility can not justify failure to accept and use present methods until or unless better are developed.

7. In the matter of new construction no conclusions can yet be formulated as to the relative merits of

building fine crushed rock or gravel surfaces, to be immediately oil-treated, or of adopting some form of bituminous macadam.

8. Protection of vehicles from freshly applied oil is indispensable; otherwise, damage to traffic may more than offset the benefits resulting from treatment.

9. Examples of oil-treated natural soil roads furnish impressive figures of good service and low costs. This type has a larger field than has heretofore been realized where soil, climate and traffic are favorable. It deserves more scientific study and active consideration.

10. *Precautions.*—The immediate improvement that generally follows oil treatment has sometimes encouraged a feeling that almost any road may be successfully treated by almost any method. Subsequent complete or partial failure has discredited the process. The facts are that success will be proportionate to the suitability of physical conditions, intelligence of supervision, and skillfulness of workmanship. The following are important precautions to be observed.

(a) Unless there is a maintenance organization accustomed to giving daily attention to necessary repairs, oiling should not be undertaken. Delay and neglect cause rapid destruction and discredit the process with the public, which expects more of an oiled than of an untreated road. The success in Oregon which has led to renewed interest in the whole subject was due in large measure to the prior development of an efficient maintenance organization.

(b) There must be sufficient thickness of metal to carry the expected traffic.

(c) The width of the stone surfacing should be adequate—rarely less than 18 feet. Vehicles turning on and off the oiled surfaces tend to crumble the edges.

(d) The metal must be compacted completely from top to bottom if the method of surface treatment is to be used, and at least through the untreated base if the surface-mixing method is to be followed. A layer of loose material between base and mixed top may cause rutting, shoving, or breaking.

(e) For surface treatment, repairs should be completed and the road compacted well in advance of oiling. Spots escaping attention should be repaired between applications of oil.

(f) All loose material and scales of fines must be removed before the oil is applied to the surface. Sweeping must expose clean rock or gravel firmly embedded.

(g) The texture of the stone surface should be uniform. Results of surface treatment will be poor if portions of the road are porous while other portions are impervious. Minor irregularities will disappear in the mixing process, but large areas of irregular grading require skillful treatment.

(h) Asphaltic materials must be applied uniformly under pressure. Streaks and omissions will be a constant source of future trouble.

(i) California asphaltic residual oil does not evaporate under atmospheric conditions, and hardens by oxidation but slowly. Therefore, excessive applications should be avoided during construction and maintenance. This precaution is now better observed during construction than formerly, but is still neglected in maintenance. Excess oil in patching causes lumps which are hard to remove. Reoiling should be done sparingly and only where there is clearly a deficiency in oil.

(j) Cover material for surface treatment should be coarse rather than fine. Clean chips are the best cover material. Fine material tends to form an unstable mat and absorbs oil which should be allowed to pene-

trate. Success with the surface-mixing method requires sufficient insert fines to fill all voids. If not already present they must be added. Fuel oil will not bind clean, coarse material.

FINE CRUSHED ROCK AND GRAVEL ROADS

The fine crushed rock and gravel roads which, since the war, have been built extensively in the Western States are different from true macadam in several particulars. The maximum mechanical bond, obtained in macadam by the interlocking of carefully sized broken rock, is not developed. The fine surfacing material is bonded under traffic, frequently with the addition of filler and binder, and sometimes with water. The usual requirement of maximum size is $1\frac{1}{2}$ inches for the base course and 1 inch or three-fourths inch for the surface course.



FIG. 1.—MIXING ROAD METAL AND BINDER, ONE OF THE PROCESSES IN THE CONSTRUCTION OF FINE CRUSHED ROCK AND GRAVEL ROADS

TYPICAL SPECIFICATIONS

The following extracts from specifications are typical:

Separation of fine from coarse material.—In the hauling of materials the contractor shall use means which will prevent, as far as possible, a separation of the fine and coarse materials. So far as possible material being transferred into stock piles, bins, or vehicles shall be dropped vertically.

Size of materials.—Materials for the base course shall all pass a $1\frac{1}{2}$ -inch circular opening and shall be well graded from that size to dust, containing not to exceed 35 per cent of material passing a $\frac{1}{4}$ -inch screen. Materials for the top course shall all pass a 1-inch circular opening and shall be well graded from that size to dust, containing not to exceed 35 per cent of material passing a $\frac{1}{4}$ -inch screen. (Some specifications limit the size to that passing a $\frac{3}{4}$ -inch opening and permit 50 per cent or more passing a $\frac{3}{4}$ -inch screen.)

Dumping on roadway.—The spreading of any course or layer of materials shall be done with suitable spreader wagons and dump trucks with gauge adjustments to give the proper spread or from piles dumped along the road. No segregation of large and fine particles will be allowed and the materials as spread shall be free from pockets of large or fine material.

Hauling, dragging, etc.—With the view of securing a maximum amount of uniformly compacted surface, the contractor will be required, as far as practicable, to do his hauling over the surfacing as it is placed; to construct the surfacing beginning at the point of loading and working away from that point; to construct the entire base course from any set-up before beginning construction of the top course from the same set-up; and to thoroughly drag the surfacing at frequent intervals during the course of the construction.

(k) Heavy clay binder mingled with the road metal greatly reduces the probability of success with oil treatment. A road that breaks up in the spring or that becomes muddy in wet weather should not be oiled.

It is common practice to require manipulation of the surfacing material with a heavy blade grader by repeatedly casting into windrows and respreading. This is for the purpose of correcting segregation of materials and improving the distribution of fines and binder uniformly throughout the mass. The operation is illustrated by Figure 1. Even where water is expensive its use during manipulation of material is frequently required and is highly advantageous.

THE COST OF THE UNTREATED ROADS

The fine crushed material generally ranges in cost from \$2 to \$3 a cubic yard, measured in trucks at the point of delivery, but has been as low as \$1.20 and more than \$5, depending upon relative availability and length of haul. The quantity used ranges from 1,200 to 2,500 cubic yards per mile; and the normal cost is, perhaps, \$4,000 to \$6,000 per mile for 18-foot width. Compared with gravel roads of the older type these surfaces are little if any more expensive and their riding qualities are often much superior. As previously stated, they cost less than water-bound macadam and are more readily maintainable.

Ordinary maintenance of this type of road is accomplished with a road drag or with a blade grader or a combination of the two. Ideally, the road should have on its surface a light mulch of about half an inch of loose, fine material to protect the compacted metal. The constant working of the fine material across the road is a most effective precaution against the formation of pits and keeps corrugations to the minimum.

Frequently the available surfacing material is deficient in cementing properties or in fines. Because of such a deficiency, many crushed rock and gravel roads, constructed a few years ago, failed to compact under traffic, and it was later found necessary to add cementitious material. More recently California and Oregon have added filler or binder during construction. Oregon frequently uses about 6 cubic yards of clay binder per 100 feet of 18-foot surface, which is approximately 25 per cent of the top course. This tends to make a firm surface and reduces the loss under traffic. A mulch is not required under such circumstances, and maintenance consists of scarifying and reworking once or twice yearly instead of systematic and continuous dragging. A road so bound becomes more or less pitted and potholed prior to each reworking, and is susceptible to damage during the spring season, particularly if the binder is a heavy clay, highly retentive of moisture.

The road metal used varies widely in characteristics. Crushed, hard rock is much preferred, but soft crushed rock, crushed gravel, and even water-worn screened gravel are used where better materials are unavailable.

It has been commonly assumed that a fine crushed rock or gravel road, without clay binder, built of average material in arid territory is satisfactory for traffic not exceeding 200 or 300 vehicles per day. With exceptionally good material, or under favorable climatic condi-

tions, its field may be extended to 400 or even 500 vehicles per day. R. H. Baldock, maintenance engineer for the Oregon highway department, estimates that a clay-bound road of this type will successfully carry 600 vehicles per day in arid territory, but opportunity to verify this estimate has been lacking.

The cost of maintenance varies widely according to traffic, climate, and character of material, but even more according to the service standards of the authorities in charge. If an average figure must be set, it may be estimated at \$1 per year per average daily vehicle where the traffic ranges from 200 to 500 vehicles per day. This covers ordinary maintenance alone; that is, the preserving of a smooth surface under traffic. It does not include periodical renewals of material.

Advantages and disadvantages of fine crushed rock and gravel roads.—Fine crushed rock and gravel roads, by systematic maintenance, may be kept smooth under modern automobile traffic more readily than surfaces composed of coarse material. The wear under traffic and weather, however, is rapid. Steps are being taken to determine definitely the wear under varied conditions—information which has not been acquired heretofore. However, various State highway departments have estimated the approximate wear by the interval between resurfacing operations and the amount of material added over a series of years, and these estimates indicate that the loss of material is large. Wyoming and Colorado report an inch of material lost per year under traffic of 200 or 300 vehicles per day and even higher losses in districts where wind is unusually severe. Wisconsin reports a loss of 1 to 1½ inches per year for traffic of 500 vehicles per day, and Oregon and California confirm the Wisconsin figures.

A loss of 1 inch of compacted material on an 18-foot road represents approximately 400 cubic yards of loose material per mile which will cost from \$800 to \$1,200 to replace.

Assuming a daily traffic of only 200 vehicles, a loss of one-half inch per year at \$2.50 per cubic yard, loose measurement, and disregarding interest, the annual cost of maintaining a mile of fine crushed rock or gravel surface may be estimated as follows: Ordinary maintenance, \$200 per year; replacement, 200 cubic yards at \$2.50, \$500; total \$700. This is \$3.50 per daily vehicle, or nearly 1 cent per vehicle-mile. This

illustration is not necessarily an average case, but it shows that the amounts spent year by year do not represent the ultimate maintenance costs.

With good road metal and a moist but not excessively rainy climate the replacement cost may be lower than above estimated. With poor material and an arid climate it will be much higher. It will be an extraordinary case where the item of replacing destroyed material is not as high as ordinary maintenance and frequently it may be expected to be two to four times as much. It is an item frequently overlooked in fixing annual budgets, accumulating as "deferred maintenance" until resurfacing is required.

Metal is removed from the road in two forms: (1) Particles thrown to shoulders; and (2) dust produced by grinding of stone on stone. A considerable portion of the loss consists of particles thrown from the road; and the dust is a menace to the traveling public. It is intensely disagreeable on a heavily traveled road; sometimes causes accidents by obscuring vision; and damages cylinders and bearings in motor vehicles, and causes serious financial loss, where the road is bordered by farms, villages, or resorts.

Another objection to the fine crushed rock or gravel type arises from excessive tire wear and rather high tractive resistance. Measurements made during the past three years by the State College of Washington² indicate that tire wear from four to five times as great as that caused by concrete or asphaltic concrete surfaces may be caused by untreated fine crushed rock surfaces upon which a loose mulch is maintained; and the same tests indicate that there is a great reduction in the amount of tire wear when the crushed rock surface is treated with asphaltic oil. The tests are not sufficiently extensive to justify acceptance of the exact ratios mentioned but, in a qualitative way, they confirm inferences that are obvious from the nature of the respective types.

Similarly, studies at the Iowa State College³ indicate that the gasoline consumption may be one-fourth more on a gravel road than upon a hard pavement. A similar relation presumably prevails between the fine crushed type and oil-treated roads.

² Engr. Bulletins, Nos. 17 and 18. Engineering Experiment Station of the State College of Washington, Pullman, Wash.

³ Bulletin No. 69. "Highway Transportation Costs," Engr. Exp. Station, Iowa State College, Ames, Iowa.

TABLE 1.—Cost of ordinary maintenance and value of metal destroyed by traffic on fine crushed rock and gravel roads prior to oiling, in Oregon and California

Project	Daily traffic	Length	Time from construction to resurfacing ¹	Ordinary main-tenance		Loss of metal				
				Total expendi-ture	Cost per mile per year	Depth	Cost per unit	Unit	Value	
									Per mile	Per mile per year
Oregon:		<i>Miles</i>	<i>Months</i>			<i>Inches</i>				
Federal Aid No. 37.....	600	12.5	45	² \$13, 976	\$372				\$4, 474	\$1, 192
Federal Aid No. 25.....	470	14.81	42	² 20, 140	453				³ 610	³ 174
Federal Aid No. 24.....	500	14.95	39	² 15, 138	338				³ 1, 260	³ 386
Federal Aid No. 42.....	450	6.80	30	² 30, 722	468				4, 911	1, 567
		15.08	41							
Morrow-Umatilla County line to Echo.....	420	19.8	51	² 21, 210	357				5, 378	1, 265
California:										
Imper al-27-B.....	1, 000	14. 07	23	4, 826	180	2. 50	2. 65	Cubic yard.....	1, 934	1, 010
Imperial-26-A.....	1, 000	3. 93	30	13, 184	1, 342	3. 48	4. 53	do.....	4, 729	1, 890
Imperial-26-F.....	2, 500	.50	15	957	1, 531	1. 88	2. 55	Ton.....	2, 156	1, 726
Imperial-26-G.....	2, 500	8. 81	10	24, 264	3, 310	2. 25	2. 55	do.....	2, 581	3, 097
South Boulevard-31-D.....	500	15. 44	6	20, 361	2, 640	.78	2. 79	do.....	979	1, 958
South Boulevard-31-E.....	275	9. 55	6	11, 326	2, 372	1. 77	2. 79	do.....	2, 222	4, 444

¹ For California projects the time is from construction to oiling.

² Three complete years, 1922, 1923, and 1924.

³ Restoration incomplete.

Fine crushed rock and gravel roads are intended for light traffic, generally not exceeding 300 vehicles per day. However, lack of funds for heavier construction or the need of temporary surfacing on a new grade prior to paving have frequently led to the use of this type for heavy traffic. When the traffic exceeds the normal volume for which the type is suitable, maintenance increases in proportion. Sprinkling is a common device introduced at this stage. Constant addition of material becomes necessary and maintenance operations seriously interfere with traffic.

Table 1 shows losses of material on several projects in San Bernardino and Imperial Counties, Calif. The loss of material is indicated by the amount of new material it was found necessary to apply to restore the original thickness of the surface when it was being prepared for bituminous treatment. Estimated in this way, no account is taken of large quantities of material

added between the date of original construction and bituminous treatment; yet it will be seen that even on this conservative basis the loss of material ranges in value from \$1,000 to nearly \$4,450 per mile per year, under traffic which in no case exceeded 2,500 vehicles per day.

To summarize: The fine crushed rock or gravel road is a type that can be built at relatively low cost; is adaptable, by the use of the requisite thickness of material, to most kinds of soil and weights of vehicle; and can be kept in smooth condition by a reasonable expenditure with simple tools and ordinary skill. There are three main objections to its use, which are, (1) that the loss of material under traffic makes it ultimately a very expensive type of surface, (2) that the dust is a serious nuisance and often costly, and (3) that the cost of operating vehicles over it is high.

SURFACE OILING OF CRUSHED ROCK AND GRAVEL ROADS IN OREGON

The extensive oil treatment program in Oregon is the outcome of an attempt in 1923 to use a grade of oil known locally as "fuel oil" as a dust palliative. A section of road was oiled at that time with one-half gallon per square yard. The section thus treated was so sticky and spattered automobiles so badly that the remainder of the road that season was given a somewhat lighter treatment. The section with the heavier application, however, turned out so well the following season that R. H. Baldock, then division engineer, now maintenance engineer for the Oregon Highway Commission, carried on extensive experiments with the idea of standardizing the treatment so that it could be economically and safely used on the hundreds of miles of crushed rock and gravel surfaces in eastern Oregon, and similar experiments were carried on in another division under the direction of C. W. Wanzer, division engineer. The experiments were regarded as successful, and the State, in response to the wishes of enthusiastic motorists, has carried on oiling as rapidly as possible on all roads in condition for treatment. Over 500 miles were reported by the commission as having been treated up to the close of 1926.

These treated Oregon highways are typical crushed stone, crushed gravel, or screened gravel roads constructed from one to four years before oiling. Usually the surface as originally constructed had been formed by the spreading of about 2,200 cubic yards of the surface metal per mile to a width of 16 to 20 feet. Normally, the base course, consisting of material passing a $1\frac{1}{2}$ -inch circular opening, constituted half the finished thickness, the wearing course, of equal depth, consisting of material passing a three-fourths-inch circular opening. Usually each course had been subjected during construction to the traffic of trucks, had been well watered, and was well bound at the completion of the project.

Most of the projects had been resurfaced before treatment to replace losses caused by traffic and weather, but the resurfacing had not in all cases restored the surfaces to their original condition. On inspection in the course of this cooperative investigation, sections were found of which the thickness was

only $3\frac{1}{2}$ inches and others of varying depths between this minimum and the original thickness of 6 inches. The subgrade of the thin sections inspected was generally rather sandy—in some cases pure sand—and this probably accounts in a measure for their ability to carry traffic.

In some of the earlier treatments the surface of the road was not broomed before applying the oil. Such surfaces observed in the course of the cooperative investigation were found to be slightly nonuniform. While quite comfortable to ride over they are not as satisfactory or as easy to maintain as the sections that have been broomed before treatment.

Brooming has been the standard practice for some time, and most of the treated roads observed had been thus prepared. The general practice in preparation for oiling now requires that the surface to be treated be bladed, sprinkled, and dragged until smooth and uniform. All holes and other irregularities are repaired; and, immediately prior to oiling, the surface is thoroughly swept with a power broom (fig. 2), supplemented occasionally by hand brooming, the sweeping being continued until all patches of loose fine material are removed and the larger pieces of rock are exposed. This was the method that had been followed in the treatment of most of the sections inspected.

CHARACTER AND AMOUNT OF OIL USED

Immediately after sweeping the majority of the sections were treated with the oil in two applications with an interval of two or three days between them to permit the first to penetrate. In general, the first application was at the rate of three-tenths, and the second at the rate of two-tenths gallon per square yard. This is the original treatment, to which it is the general practice in Oregon to add in the fall of each year a light application—at the rate of from 500 to 700 gallons per mile—to freshen the previously applied oil and seal the surface for the winter; and other light applications are made from time to time as required. This practice is referred to hereafter as reoiling. For applying the oil a pressure sprayer has been found to give more uniform distribution, although fair results have been obtained with a gravity sprinkler. (See fig. 3.)

The oil used in the early work in Oregon was purchased simply as "fuel oil," with the requirement that

¹ Throughout this investigation, the Oregon State Highway Commission, through Mr. Roy A. Klein, State highway engineer, Mr. R. H. Baldock, maintenance engineer, and Messrs. C. W. Wanzer and H. C. Smith, division engineers, has cooperated to the fullest extent by furnishing samples of materials and historical and financial data. The assistance rendered has been invaluable.

it contain at least 50 per cent of asphalt of 80 penetration, and it usually contained from 50 to 60 per cent. This oil came from various fields in California and other Western States. That most commonly used in the extensive program of 1925-26 came from California and contained from 60 to 65 per cent of asphaltic residue of 80 penetration. Its specific viscosity was usually low—ranging from 10 to 20 Engler at 122° F.—but even such light oils were found to be too viscous to be applied successfully without heating. At a temperature of from 150° to 200° F., however, they spread readily, and the requisite heating may be accomplished with steam.

During 1926 the treatment with two applications of light oil, as above described, was varied by the substitution of a heavier oil or tar for the second application in the treatment of over 130 miles of road. On these sections the first application consisted of about one-fourth gallon per square yard of the 60 to 65 per cent asphaltic oil, followed in most cases by an application of one-fourth gallon of oil containing 70 to 80 per cent of asphalt of 80 penetration. In other cases a tar was

TABLE 2.—Typical California oils used in surface treatment by the Oregon State Highway Commission in 1926

Laboratory No.	Date sampled	Used near	Specific gravity	Specific viscosity, Engler, 122° F.	Loss 325° F., 5 hours	Per cent asphalt (80 penetration)	Distillation—			
							170° to 235° C. ¹	235° to 270° C.	270° to 300° C.	Above 300° C.
20235	June 26	Baker	0.940	10.0	10.8	65.4	3.7	9.5	19.3	67.5
20236	do	do	.940	9.5	10.8	65.4	3.6	12.5	11.7	72.2
20001	June 4	Payette, Idaho	.973	84.2	4.1	73.1		2.3	8.9	88.8
20002	do	do	.973	81.1	3.3	72.3		2.5	6.9	90.6
20168	June 22	Quartz	.942	9.4	10.2	61.6		9.4	14.2	76.4
20169	do	do	.942	9.3	10.6	62.2		8.0	15.5	76.5
20492	July 16	Durkee	.945	10.6	10.7	63.4	1.4	7.1	9.7	81.8
20128	June 17	do	.944	10.5	9.8	63.3		9.7	9.0	81.3
20263	June 28	Pleasant Valley	.940	10.9	10.4	63.8	2.6	8.8	11.1	77.5
20352	July 3	do	.945	10.3	10.5	63.2		8.6	12.2	79.2
20501	July 16	do	.952	13.2	9.7	66.4	0.8	6.1	9.5	83.6
20434	July 12	Huntington	.946	11.1	11.7	61.6	2.0	6.0	10.0	82.0
20491	July 15	do	.945	10.9	10.9	63.8	1.0	6.5	10.5	82.0
20431	July 10	do	.946	13.0	12.5	62.6	2.3	8.2	11.5	78.0
20432	do	do	.946	12.0	11.0	62.0	2.1	8.1	11.3	78.5
20433	do	do	.942	10.7	12.0	60.8		7.9	13.3	78.9
20386	July 7	do	.946	10.7	10.3	63.5		9.1	13.0	77.9
20387	do	do	.944	11.0	12.1	62.9	2.0	9.3	9.5	79.2
20353	July 3	do	.944	9.5	11.2	63.5		9.9	10.9	79.2
20354	do	do	.940	10.9	11.0	63.2		10.2	9.7	80.1
20355	do	do	.940	11.8	10.1	63.2		8.2	10.7	81.1
20384	July 6	do	.940	10.2	10.9	63.6		9.1	14.0	76.9
20385	do	do	.940	8.8	12.3	61.9		9.2	13.0	77.8
20288	July 2	Haines	.940	9.2	9.7	64.5	1.0	5.5	11.1	68.1
20526	July 13	do	.975	74.8	3.9	76.5		.5	4.8	94.7
20525	July 10	Baker	.975	75.2	3.4	77.2		.5	4.9	94.6
19632	April 21	Blalock	.975	92.3	3.4	75.0		1.0	3.3	95.7
19801	May 18	Parkdale	.952	12.7	10.4	65.3		7.0	8.0	85.0
19845	May 21	do	.945	10.8	14.3	64.5		8.5	9.9	81.6
19849	May 24	Redmond	.945	10.2	11.1	61.5		5.7	10.5	83.8
19850	do	do	.944	10.2	11.6	61.8		6.0	10.0	84.0
19789	May 18	do	.943	9.8	10.2	62.9		4.8	10.2	85.0
19790	do	do	.943	9.6	9.7	64.0		5.0	9.8	85.2
19843	May 22	do	.944	9.9	9.7	62.2		5.4	11.0	83.6
19844	do	do	.945	10.2	9.5	63.2		5.2	10.6	84.2
19887	May 26	do	.945	8.8	10.1	63.0		5.0	10.3	84.7
19869	do	do	.945	10.3	9.4	62.1		5.8	10.0	84.2
19870	do	do	.945	9.8	10.2	62.8		5.8	9.6	84.6
19868	May 25	Parkdale	.933	7.9	10.8	64.3		6.5	9.0	84.5
19891	May 26	do	.943	9.4	12.7	62.8		7.7	9.3	83.0
19908	May 29	do	.942	9.0	9.4	63.8		6.3	10.0	83.7
19909	do	do	.943	10.5	10.7	61.2		8.9	9.6	81.5
19959	June 3	do	.943	9.2	9.7	60.7		9.2	9.7	81.1
20041	June 9	Rufus	.947	10.2	10.3	61.8		9.1	10.0	80.9
19910	May 28	Redmond	.945	9.2	10.8	64.0		7.6	10.1	82.3
19920	June 1	do	.945	9.1	11.0	61.5		7.7	10.7	81.6
19998	June 4	do	.943	9.2	10.0	61.3		8.0	11.1	80.6
20043	May 29	do	.944	9.3	12.4	61.5		7.7	12.7	79.6
20427	July 9	do	.946	11.6	11.6	61.6	2.0	10.5	11.1	47.6
20146	June 19	Bend	.948	12.1	11.9	64.6	1.8	9.1	12.5	76.6
20670	July 26	Blalock	.950	14.3	9.1	65.3	6.5	24.3	3.2	5.4
20566	July 21	Parkdale	.945	12.7	7.4	64.0	9.3	8.8	15.0	66.9
20671	July 26	Bend	.950	12.8	9.5	66.2	4.6	15.6	19.0	60.8
20742	July 29	Rufus	.950	13.8	10.0	66.9	5.0	12.9	17.1	16.0
20775	August 4	Biggs	.950	13.8	10.0	65.8	1.8	7.3	14.7	77.6
20743	July 29	Blogett	.950	13.8	10.0	67.0	5.0	12.8	30.0	52.2

¹ No fraction below 170° C.

used for the second application, the material being prepared from crude tar obtained in the manufacture of gas from petroleum oil. Reports received in February, 1927, indicate that these heavy oil treatments have made a good showing during the winter in sections where snowfall was heavy. Table 2 shows tests of typical oil used during the 1926 season.



FIG. 2.—Power Broom for Cleaning Road Surface Prior to Treatment



FIG. 3.—Type of Distributor Used for Application of Oil

To absorb the excess oil, chips have been applied in quantities ranging from 15 to 50 cubic yards per mile. Practice in this particular has not been standardized. Larger quantities were required on the sections treated with heavy oils and tars; and in some instances sand or screenings have been used instead of chips. After re-oiling, the use of sand or fine stone chips has been found necessary where the surface had a "sheet-asphalt" texture to prevent vehicles from skidding.

The cover material has sometimes been spread from trucks by opening the end gate an inch or so and driving rapidly over the oiled surface. In other instances mechanical spreaders have been used, as shown in Figure 4.

Maintenance in Oregon begins immediately after the final oiling. At this state, brown areas appear where there is a deficiency of oil, the result generally of the picking up of the surface by steel tires or of sandy spots in the surface. These are spotted with oil, and any depressions that may require it are filled with additional chips and oil.

Thereafter the maintenance men patrol the road, carrying with them a supply of chips and oil or premixed oil and chips. Holes or depressions are filled as soon as they appear, care being taken to use no more filling

material that is needed to bring the patch flush with the surrounding surface and avoid transforming the depression into a hump. The experience thus far indicates that a two-man patrol can cover 50 miles or more of road, performing the work described.



FIG. 4.—A MECHANICAL SPREADER HAVING WHIRLING DISKS TO DISTRIBUTE SCREENINGS OVER ROAD SURFACE

COSTS OF TREATMENT AND MAINTENANCE

The average cost of the treatment described on 343 miles of Oregon highway oiled during 1926 was \$752 per mile, exclusive of the cost of cover material. The average total cost, including cover material, was placed at \$854 per mile. These averages include the cost of 133 miles which received a first treatment—about 3,000 gallons per mile—of light "fuel oil" (about 60 per cent asphalt) followed by an equal amount of heavier oil containing about 75 per cent of asphaltic residue of 80 penetration.

An analysis of the cost of a 65.8-mile section treated with two applications of light oil is given in Table 3. This shows an average cost of \$637 per mile, exclusive of the cost of cover material. The cost of the treatment in which heavy oil is used in the second application is estimated by the engineers of the Oregon department at \$888 per mile, exclusive of cover for 1927, and in their estimates of cost they allow \$53 per mile for screenings on light oil and \$150 per mile on heavy oil.

That the cost of maintaining the oiled surfaces is not expensive is shown by Table 4, which summarizes expenditures on five sections of the Columbia River Highway from 1922 to 1926. The table was compiled

TABLE 3.—Cost per mile of oiling between Heppner Junction and Barnhart, Morrow and Umatilla Counties, Oreg., 65.8 miles

Section	Length	Supervision	Preparation of road	Application of oil	Spreading cover material	Traffic protection and miscellaneous	Cost of oil	Freight on oil	Total
Heppner Junction-Umatilla County line.....	Miles 30.6	\$29	\$115	\$53	\$47	\$38	\$214	\$90	\$586
Umatilla County line-Echo.....	19.5	22	89	59	51	40	272	137	670
Echo-Barnhart.....	15.7	33	110	61	84	61	219	128	696
Average cost per mile.....		28	106	57	57	44	232	113	637
Cost per square yard.....		0.002	0.009	0.005	0.005	0.004	0.019	0.010	0.054
Cost per gallon in place.....		.005	.019	.010	.010	.008	.041	.020	.113
Per cent of total cost.....		4.4	16.6	9.0	9.0	5.8	36.4	17.8	100

¹ Prepared from information furnished by R. H. Baldock, maintenance engineer, Oregon Highway Commission.

A total of 373,682 gallons was used, an average of 5,700 gallons per mile and 0.495 gallon per square yard.

from information furnished by the State highway commission, except that some of the reoiling cost has been rather arbitrarily included in the cost of original oil treatment. Table 5 is a summary of the data contained in Table 4.

These tables show that the cost of maintaining the untreated crushed rock surfaces on these projects, under an average traffic of 500 vehicles a day, averaged \$402 per mile per year; and that the cost of replacing the surfacing material destroyed by traffic and weather was \$1,351 per mile per year—a total of \$1,753 per mile per year, or an average annual charge of \$3.50 per mile per average daily vehicle.

The cost of maintaining these surfaces after oiling, during 1926, which was the only complete year covering all projects, averaged \$333 per mile, a figure which includes some reoiling. If the treatment is assumed to have a life of only three years, of which half on the average has already passed, \$198 (one-third of \$594, the cost of oiling) should be added to the yearly maintenance charge to obtain the total annual cost, which

TABLE 4.—Maintenance and treatment costs of Columbia River Highway in Oregon

Project	Length	Year	Average daily traffic	Untreated road costs		Treated road costs	
				Ordinary maintenance	Resurfacing to restore thickness	Oiling and reoiling	Maintenance of oiled surface
Federal aid No. 37.....	Miles 12.5	1922	309	\$4,002			
		1923	674	1,880	\$21,359		
		1924	828	8,094	34,568	\$2,059	
		1925	880			3,068	\$5,836
		1926	1,000				6,080
Total.....				13,976	55,927	5,127	11,916
Average per mile per year.....				372	1,192	410	2,486
Average per mile.....							
Federal aid No. 25.....	14.81	1922	309	2,833			
		1923	455	2,729			
		1924	640	14,578	9,034	1,920	
		1925	1,035	785		7,402	7,160
		1926	904				4,699
Total.....				20,140	9,034	9,322	11,859
Average per mile per year.....				453	1,174	628	2,317
Average per mile.....							
Federal aid No. 24.....	14.95	1922	309	1,296			
		1923	484	2,272			
		1924	728	11,570	18,782	1,099	
		1925	1,000	868		6,838	1,832
		1926	900				3,532
Total.....				15,138	18,782	7,937	5,364
Average per mile per year.....				338	386	531	2,336
Average per mile.....							
Federal aid No. 42.....	21.88	1922	309	8,577			
		1923	466	8,737	71,514	749	
		1924	585	13,408	32,460		
		1925	877	2,500	3,485	11,693	11,662
		1926	1,085				7,204
Total.....				30,722	107,459	12,442	18,866
Average per mile per year.....				468	1,587	568	2,329
Average per mile.....							
Non-Federal aid west of Echo.....	19.8	1922	300	6,500			
		1923	400	6,980			
		1924	550	7,730	106,480		
		1925	800	2,785		13,128	5,487
		1926	1,000			2,342	6,676
Total.....				21,210	106,480	15,470	12,163
Average per mile per year.....				357	1,265	781	2,337
Average per mile.....							

¹ Untreated road under traffic 3.7 years before oiling.

² Average for 1926 only.

³ For three years only, 1922-1924.

⁴ Untreated road under traffic 3.5 years before oiling.

⁵ Untreated road under traffic 3.2 years before oiling.

⁶ 0.8 mile non-Federal aid road added.

⁷ Untreated road under traffic 3.1 years before oiling.

⁸ Untreated road under traffic 4.2 years before oiling.

TABLE 5.—Analysis of surface maintenance expenditures on a portion of the Columbia River Highway in Oregon, 1922-1926, inclusive

Project	Length <i>Miles</i>	Untreated road costs		Treated road costs	
		Ordinary maintenance 1922-1924, inclusive	Resurfacing to restore thickness	Oiling and reoiling	Maintenance of oiled surface, 1926
Federal aid No. 37	12.50	\$13,976	\$55,927	\$5,127	\$6,080
Federal aid No. 25	14.81	20,140	9,034	9,322	4,699
Federal aid No. 24	14.95	15,138	18,782	7,937	3,532
Federal aid No. 42	21.88	30,722	107,459	12,442	7,204
Non-Federal aid west of Echo	19.80	21,210	106,480	15,470	6,676
Total	83.94	101,186	297,682	50,298	28,191
Average per mile per year		¹ 402	² 1,351	³ 594	³ 333
Average per mile					

¹ Average daily traffic on untreated roads, 500 vehicles per day.² Federal-aid projects Nos. 24 and 25 excluded because they were not fully restored to original thickness.³ Average for 84.7 miles, because of addition of 0.8 mile to Federal-aid project No. 42 in 1925.

is thus computed as \$531 per mile. As the traffic since oiling has averaged 980 vehicles per day, the above estimated annual cost would be equivalent to only 54 cents per daily vehicle per mile.

The costs presented in Tables 4 and 5 do not necessarily represent average conditions in the West. The use of more and better binders in the untreated surfaces, and larger expenditures for ordinary maintenance of these surfaces during the first two years would have cut the cost of replacement of untreated rock materially, although wind and dry atmosphere are particularly trying on these projects. However, there is ample evidence that the resurfacing operations did not restore the surface to its original thickness in many places, which means that the actual loss of material was greater than indicated by the cost reports. On the other hand, the operations of oil application and maintenance of the oiled surface were new to the organization, which suggests the possibility of lower costs for the same class of work in subse-

quent years. The tables do represent the actual history of the first extensive section of Oregon State highway to be treated with asphaltic "fuel oil." And they do indicate also the economy of the surface oiling as a means of salvaging these surfaces, for this fact stands out: That the cost of oiling this section was less than the annual cost of replacing lost surfacing material on the untreated roads, and the cost of maintaining the oiled surfaces is no greater than the cost of ordinary maintenance of the untreated surfaces before oiling and under lighter traffic.

INSPECTION OF OREGON OIL-TREATED HIGHWAYS

In May, 1926, during the course of the cooperative investigation here reported, an inspection was made of about 235 miles of the oil-treated crushed rock and gravel roads in Oregon, the treatment of which has been generally described in the foregoing pages. As the inspection progressed, samples were taken as representative of the surfacing materials used in the original construction, of the oil used, and of the oiled surface and base of the treated roads; and at the same time the surfaces examined were classified according to the texture and degree of displacement of the surface.

Differences in texture and surface condition were quite noticeable. A considerable mileage has a non-skid surface. Other portions have a fine, smooth texture closely resembling that of sheet asphalt, and a small percentage has a similar fine texture but is unstable and shows a noticeable tendency to creep, push, and "cabbage leaf."

The three types of surfacing were classified according to appearances as:

- A.—Non-skid.
- B.—Fine, granular, sheet-asphalt texture.
- C.—Surface showing a tendency toward displacement.

In some places sections of A and B types or of B and C types were found quite close together.

In taking samples of the surfacing material they were selected as representative of these three conditions of surface in an effort to determine by tests the reasons for the differences noted.

TABLE 6.—Tests of binder material used in Oregon

Test No.	Source	Point of use	Mechanical analysis—material passing various screens										Cementing value	Lineal shrinkage	Field moisture equivalent
			½-inch screen	No. 3	No. 10	No. 20	No. 30	No. 50	No. 100	No. 200	Removed by elutriation ¹				
		Columbia River Highway	Per cent	Per cent	Per cent	Per cent	Per cent	Per cent	Per cent	Per cent	Per cent	Pounds per square inch	Per cent	Per cent	
2407	Bluff above Seufert	Mile 94-106.44	95	95	92	90	87	83	77	67	63	226	5.7	25.2	
536	Pit above Rufus	Mile 110-121.31			99	98	97	95	91	87	75.6	212	10.8	45.1	
2411	Pit, mile 125.7	Mile 121±-127±			100	99	99	97	93	75	68	101	2.1	23.5	
2401	Arlington pit, milepost 147.4	Mile 136.27-156.62	100	90	60	42	35	31	27	20	18	68	1.7	17.6	
2412	Castle pit, mile 162.3	Mile 156.62-191+	97	96	94	91	88	88	82	72	70	114	8.8	33.1	
564	Pit 2 miles south of mile 181.7	Mile 175-190	100	99	99	97	94	91	86	80	75	125	7.4	48.6	
2408	Pit, mile 194.4	Mile 191-193+	98	98	97	95	93	91	79	69	66	94	8.8	30.8	
561	Pit, mile 199	Mile 197-201		100	99	97	96	94	91	86	88	102	5.7	57.8	
562	Pit, mile 203	Mile 201-208	100	92	65	35	26	23	20	18	15.7	54	0.0	11.8	
563	Pit, mile 211.5	Mile 208-214			100			98	96	94	81.2	198	5.3	29.5	
2415	Stannel pit, mile 220	208 to 224+	100	99	99	98	97	96	95	93	92	103	3.3	22.9	
2413	Campbell pit, mile 241.49	Old Oregon Trail	100	99.2	96.4	94.8	93.6	92	89.6	85.4	83.0	193	8.4	32.0	
2410	Bad Roads pit, mile 245.56	do		100	99	94.8	94.4	82	76.8	71.8	70.6	165	4.7	23.9	
2420	Pit 3 miles east of Kamela, about mile 271	do	100	98.4	75.2	57.6	53.6	49.4	47.4	36.4	36.0	135	2.2	22.5	
2419	Pit 5 miles east of Redmond	The Dalles-California Highway, mile 115-140			100	99.8	99.3	97.9	96.3	95.1	94.6	260	9.6	31.7	

¹ See Department Bulletin 1216, U. S. Department of Agriculture, p. 10, Clay and Silt Test.² Binder in pit mixed with gravel.

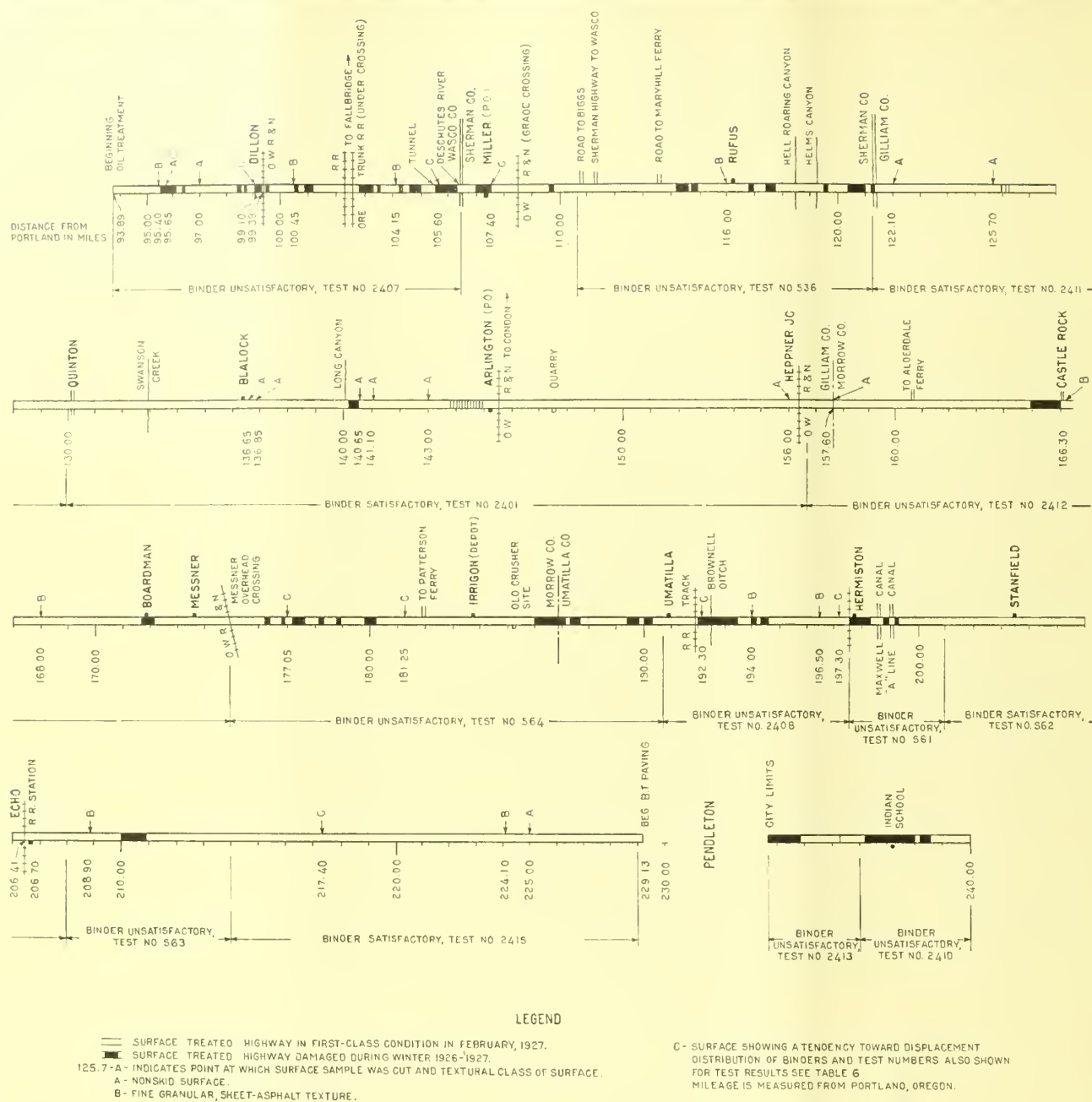


FIG. 5.—CONDITION OF OILED SURFACES ON COLUMBIA RIVER HIGHWAY FROM A REPORT ON AN INSPECTION MADE IN FEBRUARY, 1927

The condition of the various sections of the Columbia River Highway between The Dalles and Pendleton, the classification of the surface according to texture at a number of points, and references to tests of samples taken from these points are shown in Figure 5. Figures 6 to 19 show the appearance of the surface at various points (identified with the chart by distance in miles from Portland) and indicate the generally satisfactory condition of the oiled surfaces.

The results of physical tests of the mineral binder contained in samples of the crushed rock taken at various points along this highway are given in Table 6; and tests of samples of the oil-treated surface, showing the mechanical analysis of the crushed rock and the character and amount of the oil, are reported in Table 7.

Several outstanding facts appear to be established by these tests.

The A samples were found to contain the least average amount of bituminous material. However, the variation in the amount in the three types of surface is apparently slight, and can not be the sole cause of the physical differences visible on the road. (See Table 7.)

It is significant that the A, or nonskid, type of surface contained the least proportion of mineral binder, and that the binder present had relatively low water carrying capacity and a low shrinkage value. (See Table 6.)

The proportion and character of the clay and silt binder appear to be the principal difference between the A and B samples as revealed by analysis. It is interesting to note (Table 6) that, although the texture of

TABLE 7.—Analyses of surface samples from surface-treated sections of the Columbia River Highway

Distance of sampled section from Portland	Texture of surface	Test No.	Analysis of total sample		Analysis of material passing 10-mesh sieve						Number of washings to remove clay	Oil as a percentage of 10-mesh material	Bituminous content				Sample		
			Passing 1/8-inch screen	Passing 10-mesh sieve	10-mesh sieve	20-mesh sieve	50-mesh sieve	100-mesh sieve	200-mesh sieve	Clay			Evaporation 325° F.	By extraction	Per cent of total sample	Depth	Thickness	Oil in surface	
			Per cent	Per cent	Per cent	Per cent	Per cent	Per cent	Per cent	Per cent	Per cent	Per cent	Per cent	Per cent	Per cent	Ins.	Ins.	Gals. per sq. yd.	
95.65	A	18	83.2	32.4	100	78.6	61.5	48.4	36.5	12.3	24	2.8	0.3	0.6	0.9	Ins.	Ins.	0.106	
97.0	A	20	58.1	37.2	100	92.0	76.9	55.3	40.6	17.9	16					0-1.5	1.5		
		26	92.1	35.1	100	79.5	60.6	47.6	36.3	13.0	39	2.6	.4	.5	.9	Base	2.0		
122.1	A	30	93.4	54.5	100	79.3	63.8	51.9	42.1	18.2	16					0-625	.625	.067	
		1	73.4	40.9	100	81.0	63.6	55.7	33.1	6.6	12	5.4	.9	1.3	2.2	625-3	2.375		
125.7	A	22	73.4	52.9	100	92.4	80.8	72.6	36.6	6.8	10	.4	.1	.1	.2	0-1	1.0	.262	
		19	65.0	42.7	100	80.2	63.1	53.0	34.9	6.3	16	5.4	.5	1.8	2.3	1-4.5	3.5		
136.65	A	10	44.3	29.2	100	78.5	69.5	61.5	42.3	14.2	12	.6	.04	.15	.19	0-1	1.0	.274	
		4	97.7	44.2	100	77.2	57.1	45.0	27.9	6.2	13	9.5			4.2	1-3.5	2.5	.056	
136.85	A	25	88.9	60.8	100	82.6	65.8	51.6	32.1	8.8	17	1.8	.5	.6	1.1	0-75	.75	.375	
		27	89.3	41.9	100	81.3	58.1	42.8	25.8	9.3	28	6.7	1.4	1.4	2.8	.75-3.5	2.75		
		2	96.3	61.0	100	82.7	60.0	45.4	27.1	7.6	11	3.7	1.2	1.1	2.3	0-75	.75	.250	
140.65	A	29	64.8	33.4	100	78.5	62.1	48.6	29.0	9.4	13	3.0	.4	.6	1.0	.75-1.5	.75	.204	
		9	79.8	29.8	100	85.5	66.7	54.0	35.6	8.1	24	4.6	.5	.87	1.37	1.5-3.5	2.0	.238	
		8	94.4	55.2	100	88.3	68.2	53.5	34.6	9.7	20	3.8	.8	1.3	2.1	0-375	.375	.0612	
141.1	A	7	71.5	28.9	100	81.7	64.2	55.6	42.3	14.2	12	7.0	.7	1.3	2.0	.375-1	.625		
		5	73.9	25.9	100	83.3	64.4	50.8	30.1	8.2	14	23.5	2.1	4.0	6.1	1-3	2.0	.476	
		13	91.4	48.3	100	84.9	64.6	52.3	32.5	9.6	42	3.9	1.0	.9	1.9	0-125	.125	.091	
		11	90.4	61.7	100	80.5	59.5	48.7	33.0	11.3	24	3.2	.8	1.2	2.0	.125-.75	.625	.141	
143.0	A	15	70.3	43.8	100	84.3	68.4	54.9	28.6	6.3	14	4.7	.8	1.2	2.0	.75-1.5	.75	.179	
		3	97.2	36.5	100	87.0	65.5	44.8	31.2	8.2	12	6.6			2.4	1.5-4	2.5	.593	
156.0	A	36	71.2	35.2	100	75.1	55.0	42.2	30.0	11.3	26					0-1	1.0	.285	
		14	87.7	43.9	100	83.6	68.0	56.1	34.3	10.2	18	3.2	.3	1.1	1.4	1-4.5	3.5		
157.6	A	37	94.1	57.4	100	75.1	55.0	42.2	30.0	11.3	16					0-.75	.75	.125	
		17	82.0	45.5	100	84.3	74.2	61.6	37.9	11.5	44	3.9	.9	.9	1.8	.75-5	4.25		
225.0	A	6	78.3	51.8	100	87.9	66.0	57.6	35.6	15.0	15	2.0			1.1	0-1	1.0	.214	
		32	82.4	38.2	100	69.6	47.2	38.0	30.3	9.6	28	6.5	1.3	1.2	2.5	1-4	3.0	.392	
95.4	B	24	72.2	49.3	100	88.3	73.9	64.5	41.7	10.6	13	3.7	.8	1.0	1.8	0-1	1.0	.297	
		8	87.1	35.0	100	83.0	66.0	51.4	39.2	13.6	36	8.6			3.0	1-5	4.0	.853	
100.45	B	19	73.5	33.2	100	86.4	71.0	56.4	44.5	20.4	23					0-1	1.0	.356	
		1	98.3	54.6	100	74.8	52.3	42.9	34.2	13.4	29	2.4	.8	.5	1.3	0-1.5	1.5		
104.15	B	18	61.5	31.3	100	81.6	64.8	54.7	44.0	20.0	16					1.5-4	2.5	.232	
		2	97.4	60.9	100	75.8	54.0	42.0	27.8	7.9	20	6.0	2.5	1.2	3.7	0-1	1.0	.441	
116.0	B	25	71.4	36.1	100	74.0	56.0	43.7	32.3	12.7	9					1-4	3.0		
		30	91.1	42.3	100	72.8	50.7	41.8	33.1	9.1	28	7.7	1.0	2.3	3.3	0-0.5	.5	.197	
166.3	B	27	81.2	53.3	100	79.8	54.2	45.1	37.7	16.7	20					.5-3	2.5		
		6	91.2	45.5	100	81.6	65.1	50.6	37.8	13.6	43	3.0	.7	.7	1.4	0-0.5	.5	.084	
		9	64.2	37.0	100	90.2	75.3	56.7	45.5	22.4	47	4.6			1.7	.5-2	1.5	.304	
168.0	B	31	39.0	30.5	100	93.4	72.5	43.3	30.9	13.2	24					2-4	2.0		
		22	71.9	28.5	100	76.3	57.5	45.2	35.2	15.2	38	17.2			4.9	0-0.75	.75	.437	
194.0	B	17	67.4	47.8	100	90.6	78.9	60.0	48.8	25.0	18					.75-4	3.25		
		5	86.6	41.3	100	82.5	61.0	45.5	34.4	12.0	42	11.6			4.8	0-1.25	1.25	.714	
196.5	B	26	86.5	72.9	100	91.1	74.1	36.0	19.3	9.9	9					1.25-4	2.75		
		3	80.4	39.3	100	78.8	60.0	49.9	41.4	17.1	39	13.7	1.2	4.1	5.3	0-0.75	.75	.473	
208.9	B	23	82.3	47.5	100	87.8	75.9	67.1	56.0	26.5	24	2.5	.4	.8	1.2	.75-2	1.25	.179	
		20	83.2	64.2	100	95.0	61.8	40.1	31.1	11.4	8					2-4	2.0		
224.1	B	4	85.9	44.0	100	80.1	63.2	56.2	50.4	19.0	42	13.4			5.9	0-75	.75	.527	
		24	84.1	48.4	100	89.2	81.8	77.8	72.2	31.0	15					.75-5.75	4.25		
95.65	C	7	75.9	42.0	100	84.9	60.0	51.8	42.2	13.2	40	7.1			3.0	0-1	1.0	.356	
		16	67.7	36.8	100	85.2	68.3	58.2	49.5	19.5	11					1-4	3.0		
99.1	C	25	77.4	38.3	100	76.4	56.3	42.2	32.2	11.2	33	11.5	.6	3.8	4.4	0-1	1.0	.524	
		24	87.1	24.1	100	81.6	68.9	59.5	49.9	20.4	47	2.0	.1	.4	.5	1-2	1.0	.050	
		4	71.8	20.9	100	84.6	75.9	68.9	59.8	27.8	30	12.0	.8	1.7	2.5	2-3	1.0	.298	
105.6	C	14	95.0	34.4	100	71.4	55.7	42.3	30.0	9.4	35	6.3	.5	1.6	2.1	0-1	1.0	.250	
		6	89.4	26.0	100	85.4	73.8	59.2	45.7	19.7	30	7.7	.1	1.9	2.0	1-2	1.0	.238	
		5	67.8	50.4	100	93.2	80.4	59.0	42.5	18.7	26					2-4	2.0		
107.4	C	3	95.5	46.8	100	73.0	48.4	37.5	29.0	11.6	25	7.9	.8	2.9	3.7	0-1.25	1.25	.546	
		23	95.1	39.4	100	77.3	54.6	43.2	33.5	11.7	57-63-33	1.0	.2	.2	.4	1.25-2.5	1.25	.059	
177.05	C	27	74.6	41.3	100	87.4	72.8	60.6	49.6	21.6	27					2.5-4.5	2.00		
		2	92.6	48.2	100	75.4	51.1	40.2	31.4	11.4	18	10.8	.4	4.8	5.2	0-1.5	1.5	.923	
181.25	C	26	92.7	47.4	100	68.5	45.3	35.4	27.9	11.5	9					1.5-4.5	3.0		
		11	91.4	43.5	100	83.4	61.3	44.0	30.4	10.2	41	3.9	.9	.8	1.7	0-1	1.125	.225	
192.3	C	12	82.6	38.5	100	78.4	54.0	40.4	30.7	12.2	46	1.5	.4	.1	.5	1.125-5	3.875	.275	
		R1	84.9	36.4	100	78.4	54.0	40.4	30.7	12.2	46	1.5	.4	.1	.5	1.125-5	3.875	.275	
		R4	84.9	37.4	100	84.6	60.9	35.6	20.3	7.8	26	3.3	.5	.7	1.2	0-25	.25	.036	
		R5	61.1	28.7	100	90.4	69.0	40.8	24.0	8.6	40	3.5	.7	.6	1.3	.25-1	.75	.116	
		R4	61.1	28.7	100	88.7	69.6	50.2	35.1	15.4	40	1.4	.3	.06	.4	1-4.75	3.75	.187	
192.3	C	R3	75.6	25.2	100	76.4	50.3	36.9	26.6	9.8	51	7.5	.6	1.3	1.9	0-25	.25	.566	
		R2	72.7	33.7	100	79.3	60.7	51.4	43.0	17.4	51	1.2	.3	.09	.4	0.25-2	1.75	.084	
		R6	73.7	36.1	100	76.0	41.9	30.7											

the A surface was classed as nonskid, it actually contains more sand passing a 10-mesh sieve than the B or C types. Removal of fine sand and binder by thorough sweeping prior to oiling probably contributed to some extent to produce the nonskid surface.

In explanation of the apparent

TESTS OF BINDER USED ON COLUMBIA RIVER HIGHWAY

To determine the character of the binder used in the various sections of highway, samples taken were subjected to three tests used in recent years by the testing and research laboratory of the California Highway Commission. These include two of the simplest soil tests which were found to be generally indicative of the suitability of binder material. These are the lineal shrinkage test and the field moisture equivalent test,⁵ as described in detail by A. C. Rose of the United States Bureau of Public Roads in the issue of PUBLIC ROADS for August, 1924, except that the soil is dried at 212° F. in preparation for the moisture equivalent test.



FIG. 6.—MILE 96 ON COLUMBIA RIVER HIGHWAY. SURFACED IN 1921, RESURFACED AND OILED IN 1924



FIG. 7.—MILE 106 ON THE COLUMBIA RIVER HIGHWAY. SURFACED IN 1921, RESURFACED AND OILED IN 1924, AND REOILED IN 1925

It has been demonstrated in the soil studies that low moisture-carrying capacity and low lineal shrinkage are desirable characteristics. Such soils usually make good natural roads, except those too sandy to compact. Soils which have high moisture capacity absorb excessive quantities of water and when wet lose most of their cementing or binding property. A high lineal shrinkage indicates excessive fineness or colloidal properties, and such a soil has little road value when in a saturated condition. Such material, when mixed with road metal, can not add stability or wearing qualities except when dried or when liberally adulterated with fine sand or inert rock dust.

In addition to the soil tests the binder samples were also tested by a test used during the past three years in the testing and research laboratory of the California Highway Commission to measure the cementing property of the material. For the purpose of this test the

samples of surfacing material were screened to segregate the material passing a one-half-inch circular opening. This portion of the sample was mixed with water to the consistency of stiff concrete and tamped into 4½-inch cubical molds. After drying to constant weight at atmospheric temperature the resulting specimens were capped with plaster of Paris and broken in a standard compression machine, the average compressive strength per square inch as determined from three specimens being taken as the measure of the cementing value of the material. In its use of this test the California laboratory has found that there is wide variation between the results obtained with different materials, but fairly uniform results are had from retests of the same material. The strength ranges from 0 to 225 pounds per square inch; and a unit strength of 100 pounds has been established as the criterion of satisfactory binder.

In general, it has been found that at least 15 per cent of the material passing a one-half-inch circular opening should pass the 100-mesh sieve, and 8 to 10 per cent



FIG. 8.—MILE 116 ON COLUMBIA RIVER HIGHWAY. SURFACED WITH CRUSHED GRAVEL BOUND WITH CLAY IN 1924. LOOSE UNSWEPT SURFACE TREATED WITH 0.24 GALLON PER SQUARE YARD IN 1924. REOILED IN 1925 AT A RATE OF 0.42 GALLON PER SQUARE YARD



FIG. 9.—MILE 126 ON COLUMBIA RIVER HIGHWAY. RESURFACED WITH CRUSHED ROCK BOUND WITH CLAY IN 1924. OIL APPLIED AT A RATE OF 0.5 GALLON PER SQUARE YARD TO UNSWEPT SURFACE IN 1925. SURFACE IS NOT ROUGH BUT LACKS UNIFORMITY

should be removed in the washing test to establish satisfactory cementing property. A smaller proportion of fines, except in very soft rock, will usually result in a loose, uncompacted surface.

In addition to testing in this way the samples of crushed rock and gravel taken from the surface of the

⁵ The relation of this test to the standard moisture equivalent test is discussed by Dr. Charles Terzaghi in PUBLIC ROADS, vol. 7, No. 8, October, 1926.



FIG. 10.—MILE 136 ON COLUMBIA RIVER HIGHWAY. RE-SURFACED IN 1924, USING CEMENTITIOUS GRAVEL FOR SURFACE COURSE. OIL APPLIED IN 1925 AT A RATE OF 0.5 GALLON PER SQUARE YARD. THE SURFACE IS UNIFORMLY SMOOTH



FIG. 11.—MILE 146 ON COLUMBIA RIVER HIGHWAY. RE-SURFACED WITH CRUSHED CEMENTITIOUS GRAVEL IN 1925. OILED IN THE SPRING OF 1926



FIG. 12.—MILE 176 ON COLUMBIA RIVER HIGHWAY. RE-SURFACED IN 1923 AND OILED IN 1925



FIG. 13.—MILE 185 OF COLUMBIA RIVER HIGHWAY. RE-SURFACED IN 1923 AND OILED IN 1925. THE SURFACE BECAME BADLY RUTTED AND IN APRIL, 1926, A TAR TREATMENT WAS APPLIED WITHOUT APPARENT BENEFIT. A SURFACE SAMPLE SHOWED AN EXCESS OF CLAY SATURATED WITH MOISTURE

roads inspected, the cementing test was also applied to samples of binder which had been mixed with these roads during construction, obtaining the samples directly from the pits from which the construction supplies were taken. The method of testing in this case was similar to that previously described except that clean concrete sand was mixed with the sample of binder in lieu of the fine rock or gravel with which it would be mixed in the road. In preparing the test specimens, binder material was added to the clean concrete sand in amount sufficient to bring the fines passing a 100-mesh sieve up to 15 per cent and the amount removed by washing to about 10 per cent of the mixture. As clean concrete sand has no cementing value, the strength of the specimens measures the cementing properties of the binder material; and separate binder tests when compared with tests of the same binder mixed with surfacing material indicate that the method gives a fair measure of the additional strength contributed by the binder.

The results of these three tests—lineal shrinkage, field moisture equivalent, and cementation—together with mechanical analyses of the material are given for samples of the binder used in the Columbia River Highway between The Dalles and Pendleton in Table 6.

In practically every instance an excess of clay was found in samples taken from surfaces which were

pushing or corrugating. No such defects were found in any of the sections which had been bound with volcanic ash soil or with fines obtained in quarries or gravel pits.

If the limits suggested for the field moisture equivalent and lineal shrinkage tests (i. e., a field moisture equivalent percentage of 20 and a lineal shrinkage value of 5) and a cementing value of 100 pounds per square inch be used as criteria of satisfactory qualities in the binder, it will be noted that the samples represented by tests 536, 561, 563, 564, 2407, 2408, 2412, 2413, and 2419 would be classed as unsatisfactory. The material represented by test 2410 would be regarded with suspicion and would probably be rejected because of the high moisture capacity, the lineal shrinkage being so near the limit; and only the material represented by tests 562, 2401, 2411, 2415, and 2420 would be considered suitable for use. Figure 20 shows typical shrinkage test specimens from these tests after drying.

Figure 5 shows the points where the samples of surfacing material were taken from the highway, and the limits between which the binder materials represented by the tests reported in Table 6 were used. It also shows the location and extent of failures in the oiled surface resulting from frost and precipitation during the last winter, as reported by A. F. Morris, associate highway engineer of the Bureau of Public Roads, under



FIG. 14.—MILE 156 OF COLUMBIA RIVER HIGHWAY. RESURFACED WITH CRUSHED CEMENTITIOUS GRAVEL IN 1925 AND TREATED WITH 0.5 GALLON PER SQUARE YARD OF OIL IN THE SPRING OF 1926



FIG. 16.—MILE 196 ON COLUMBIA RIVER HIGHWAY. RESURFACED IN 1924. OIL APPLIED AT A RATE OF 0.495 GALLON PER SQUARE YARD IN 1925. SURFACE RECENTLY REOILED LIGHTLY AND SANDED



FIG. 15.—MILE 166 ON THE COLUMBIA RIVER HIGHWAY. RESURFACED IN 1923. TREATED WITH OIL AT A RATE OF 0.495 GALLON PER SQUARE YARD AND STONE SCREENINGS AT A RATE OF 15 TO 20 CUBIC YARDS PER MILE IN 1925



FIG. 17.—MILE 206 ON COLUMBIA RIVER HIGHWAY. RESURFACED IN 1924 AND TREATED WITH OIL AT A RATE OF 0.495 GALLON PER SQUARE YARD IN 1925. NOTE THE TRAFFIC GUIDE LINE AS USED UPON HIGH-TYPE SURFACES

date of February 11, 1927. It will be noted that the sections bound with suitable binder, 57.5 miles, show only one mile of broken surface; whereas sections surfaced with material rated as "unsatisfactory," 85.8 miles, show 16.8 miles broken.

The failures reported do not imply that the road is unserviceable, but merely that the bituminous surface has broken or potholed to the extent of requiring partial retreatment during the coming season.

The subsoil conditions on these eastern Oregon highways are generally favorable, the subgrade being sand or sandy soil. If the subgrade had consisted of material having high moisture carrying characteristics, it is quite likely that the failures would have been much more extensive and complete on the sections bound with heavy clay binder.

Clay binder represented by test 2419 (Table 6) was used on about 25 miles of The Dalles-California Highway between the Crooked River and Bend. This section was surfaced in 1921 with fine gravel containing little or no cementitious material. The surfacing did not compact under traffic, and in 1923 and 1924 binder was mixed with the gravel at the rate of from 5 to 8 cubic yards per hundred feet of road. The surfacing then compacted readily. In 1926 the road was treated with light "fuel oil" at the rate of 0.408 gallon per square yard. The surface remained in good condition

during the summer and fall, but T. M. Davis, associate highway engineer of the Bureau of Public Roads, reported in February, 1927, that there had been a general breaking up following alternate freezing and thawing during the rather severe winter. Mr. Davis reported that the destruction of the surface had been practically complete where drainage conditions were unfavorable.

A study of test 2419 shows the binder to be a heavy, unstable clay, as evidenced by the high field moisture equivalent value and excessive lineal shrinkage. The oil surface prevented the escape of moisture by evaporation, and the freezing of this accumulated moisture was followed by a general breaking up of about half of the section. Figures 21 and 22 are typical views of this section before and after the winter. Comparison of the condition of surfaces bound with suitable and unsuitable binder gives strong emphasis to the importance of the binder as an element in the success of oil treatment.

TESTS OF OIL-TREATED SURFACING FROM COLUMBIA RIVER HIGHWAY

Table 7 shows the results of tests of the samples taken from the oil-treated surfacing of the Columbia River Highway at various points indicated by reference to the mileage from Portland corresponding to the



FIG. 18.—MILE 216 ON COLUMBIA RIVER HIGHWAY. RE-SURFACED IN 1923 AND OILED IN 1925

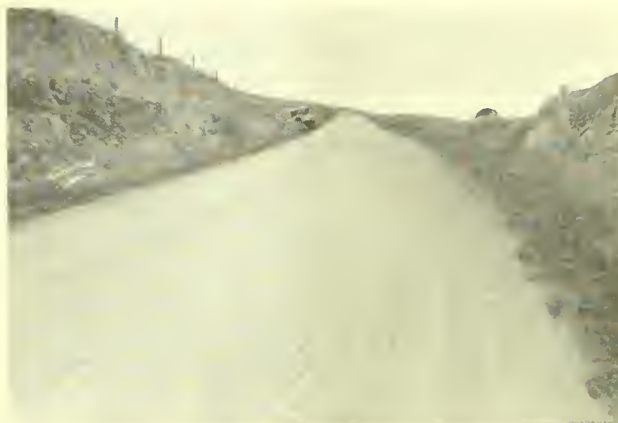


FIG. 19.—MILE 226 ON COLUMBIA RIVER HIGHWAY. RE-SURFACED AND TREATED WITH OIL AT A RATE OF 0.24 GALLON PER SQUARE YARD IN 1923 AND AN ADDITIONAL TREATMENT OF 0.26 GALLON PER SQUARE YARD IN 1924



FIG. 20.—TYPICAL SPECIMENS FROM TESTS TO DETERMINE THE LINEAR SHRINKAGE OF BINDER MATERIAL



FIG. 21.—CONDITION OF OILED SURFACE ON MILE 130 OF THE DALLES-CALIFORNIA HIGHWAY IN AUGUST, 1926



FIG. 22.—MILE 115.8 ON THE DALLES-CALIFORNIA HIGHWAY SHOWING SURFACE BROKEN UP. A SECTION CAN BE SEEN IN THE DISTANCE WHERE ONLY THE EDGES ARE BROKEN

mileage designation in Figure 5. The character of the surface represented by the samples is indicated in each case by the letters A, B, or C, referring to the surface types previously described.

In these tests the samples were first dried and then subjected to a constant temperature of 325° F. until loss had practically ceased. The purpose of this was to determine the amount of bitumen volatile at 325° F. The remainder of the bitumen was then extracted, using benzol as a solvent. The oils used in these treatments were all residues produced as by-products of distillation at high temperatures, and the tests as tabulated seem to indicate that the oil does not lose any considerable portion of volatile fractions by evaporation after exposure for a considerable period on the road. The apparent drying on the roadway is probably due to absorption and not evaporation.

After the bituminous material had been removed from the sample, the clay content was determined and a mechanical analysis was made of material passing a 10-mesh sieve. The clay determination was made in accordance with A. S. T. M. tentative specification D137-22 T, except that suspension clay was not separated. Clay as reported in the tests includes suspension clay.

The number of washings required to remove the clay is significant. Where suitable binder materials had been used, relatively few washings were required. Where heavy, sticky clay binder had been used, the number of washings was greatly increased.

The samples were taken in strata to determine the depth of penetration and the character of material in the roadway, and at the time of sampling notes were made as to condition of the road and any unusual features. Following are descriptions of a few typical samples:

Sample 99.1: This sample represents a class C surface, i. e., one showing a tendency toward displacement. The field notes read: "Layer 0 to 1 inch rich and loose. Layer 1 to 2 inches apparently clayey with some free oil and water. Layer 2 to 4 inches, excessive binder. Road has pushed and rutted but is fairly comfortable to travel." The analysis shows about equal oil content in the upper two strata, each 1 inch thick. The amount of bitumen averages 7 per cent of the 10-mesh sand or 2 per cent of the total wearing surface. Clay and silt in the upper inch are about normal for an A-type surface, but the two lower strata apparently contain about 19 per cent of clay and 25 per cent of silt. The total amount of oil found in the upper 2-inch stratum equals approximately one-half gallon per square yard, and the lack of stability in the wearing surface is not believed to be due to an excess of oil. The high clay content was probably responsible for the displacement noted. Clay for binder on this section was obtained from the Seufert pit, represented by test 2407, Table 6.

Sample 122.1: This represents a class A surface. The field notes read: "Rocky surface is typical of the John Day-Quinton section. Coarse rock in the surface stands up and gives nonskid effect. Oiled surface 1 inch thick." The analysis shows a bituminous content of 5.4 per cent of the 10-mesh material, which is about normal for A-type surfacing. The clay content is unusually low; otherwise the grading is normal. From the standpoint of nonskid qualities as well as smoothness, the section is probably ideal. The binder came from the pit at mile 125.7, test 2411 (see fig. 5 and Table 6). Only 12 washings were required to remove the clay from this sample.

Sample 192.3: Another C surface. The field notes read: "Irrigation ditch parallels road about 40 feet from the center line. Surface is rutting and pushing badly. Has been treated with tar recently on old oil applied in 1925, and the tar cake is one-fourth inch thick. From one-fourth to 2 inch depth, surfacing filled with soft, wet clay. Under this, to a depth of 4 inches, gravel bound with sandy material. Failure evidently due to excess of clay binder in upper 2 inches of road." The analysis shows a high percentage of undesirable clay binder in the wearing surface. Fifty-one washings were necessary to remove this clay. Thirteen per cent of clay was found in the base course, but the difference in the character of the clay in the base is revealed by its removal in 17 washings. In this instance, water evidently reached the surfacing by seepage from the irrigation canal and the oil mat on the surface of the road prevented the moisture from escaping by normal evaporation. Subsoil conditions were not unfavorable. The base course, as noted, was sandy. This sample and the condition of the road emphasizes the importance of the character of the binder in a roadway which is to be oiled.

Sample 224.1: This sample represents a class B surface i. e., one having a fine, granular, sheet asphalt texture. The field notes read: "This sample is from a section which was oiled in September, 1923, using one-half gallon per square yard of roadway. Texture of surface smooth and uniform. Oil surface 1 inch thick and very hard. Base from 1 to 4 inch depth bound tight." The analysis shows a very low bituminous content and a fairly high percentage of binder in the top inch. The base course, from 1 to 4 inch depth, appears to have been bound with a volcanic ash similar to that represented by test 2415. (Table 6.) This is one of the earliest oil sections constructed in the State of Oregon.

The study of the tests of surface-treated material seems to show quite conclusively that stability is best obtained where the road is well bound under traffic without the use of sticky, heavy clays. The amount of bituminous material required is small. It is quite apparent that stability in the surfacing is more dependent upon the stable character of the road material than upon the amount or kind of oil used.

SURFACE OILING OF CALIFORNIA STATE HIGHWAYS

The use of "fuel oil"—an oil with an asphalt content of 40 to 50 per cent—for dust palliative purposes was begun on California State highways in 1924. The treatment at that time consisted of an application of one-tenth to one-seventh of a gallon per square yard repeated two or three times during a season. On many sections this repeated oiling, even with light oil, tended to produce a hard, oil-bound surface. The surface thus formed was not uniform and it was necessary to keep a thin covering or mulching of oil and sand or fine gravel over the surface of the road.



FIG. 23.—POWER BROOM ATTACHED TO FRONT END OF A TRACTOR USED FOR CLEANING SURFACE BEFORE APPLICATION OF OIL

Somewhat heavier oiling was undertaken on the Redwood Highway in Division I under T. A. Bedford, division engineer, in 1925. This division is composed of the coast counties in the northern part of the State, including Humboldt and Mendocino, and the 1925 work was done in these two counties. These counties are mountainous, heavily timbered, and have a rainfall of 40 to 80 inches per season; but there is comparatively little frost and the summers are hot and dry. The dust nuisance on the Redwood Highway, which had previously been surfaced with crushed stone, was particularly objectionable where the highway passed through the orchards, vineyards, and the redwood forests. Short sections of the highway were given applications of oil in varying amounts with little or no preparatory treatment of the road. In some cases as much as one-third of a gallon per square yard was applied, and the results, from a dust-palliative standpoint, fully justified the expenditure.

In the spring of 1926 it was found that the oiled surface had withstood the winter and was in good condition in many places. The results of the applications made during the preceding season were so encouraging that the work was undertaken on a much larger scale, with the idea of attempting to produce a more lasting dustless surface. The sections selected were in typical condition; some were in a good state of preservation and others were worn to the danger point. The traffic averaged about 1,000 vehicles per day during the summer and somewhat less in winter.

From his previous experience the division engineer had concluded that sweeping prior to treatment would be very desirable, but, because of delays in obtaining the necessary equipment, this practice was not instituted until June 15, 1926. In general, the sections

which were swept prior to oiling are now in the best condition.

On the sections completed prior to June 15, 1926, the preparation consisted of the blading of the mulch into rows along the shoulders of the highway. Oil was then applied at the rate of 0.44 gallon per square yard and the loose material bladed back over the freshly oiled surface. The mulching averaged about 1 inch in thickness and served as a blanket which absorbed the oil. While this method had the merit of giving traffic entire protection against the spattering of fresh oil, the treatment was open to the very serious objection of producing a rather thin mat of oil-treated material which was not properly traffic bound. Beneath this mat there was found in practically every case examined a layer of loose, dry, unsaturated screenings and dust.

On the work carried on after June 15 the blading of the mulching into windrows was followed by thorough sweeping with a power broom attached to the front end of a tractor as shown in Figure 23, and the brooming was continued until the loose material was removed and the surface of the rock exposed, as shown in Figure 24.

After the loose material had been swept from the surface of the road, oil was applied in two applications of approximately one-quarter of a gallon each, as shown in Figure 25, the second following within a few minutes of the first. The mulching was then bladed back over the fresh oil, usually within 10 to 30 minutes after the spreading. Figure 26 shows the same section of road after the loose material had been bladed half-way across the freshly oiled strip. The next passage of the blade completed the covering.



FIG. 24.—EXPOSED ROCK SURFACE UPON COMPLETION OF BROOMING

Later in the season the dust was screened by hand from the mulching before spreading upon the oil.

On certain sections the traffic had bound the surfacing material so thoroughly that a negligible amount was removed by the sweeping. Even when supplemented by additional screenings, the cover upon the oil was very thin and was composed of relatively clean material.

About March 1, 1927, the sections treated during 1926 were examined for differences due to cover material. Although comparisons were somewhat obscured by other variables, it was evident that the well-bound sections covered with a minimum of material were in best condition, that removing dust from the mulching

was an advantage, and that sections improperly swept before oiling were distinctly inferior to those thoroughly cleaned.

OIL TREATMENT AS AN EMERGENCY MEASURE

On a number of sections in Division I the metal was known to be too thin for satisfactory treatment and was lacking in the uniformity which is required for best results. The treatment in these cases was given to prevent anticipated destruction during the subsequent rainy season. The road was bladed to remove the rough spots, and was treated with a heavy grade of "fuel oil" at the rate of a half-gallon per square yard. When the oiling was completed, the rough spots and holes were patched with premixed oil and screenings. Results have entirely justified the expenditure. In general, the sections are much smoother than they were prior to treatment, and the application of the oil has had the effect of waterproofing the road, of stopping the loss of metal, and, in many cases, of preventing a break-up during the winter.



FIG. 25.—APPLICATION OF OIL TO CLEANED SURFACE



FIG. 26.—MULCHING PARTLY BLADED BACK OVER OILED SURFACE

While oil treatment without proper preparation of the surface is not generally recommended, many cases will be found where such use of oil will be economical. Failures in such instances should not discredit oil treatment. Maintenance costs on surfaces which are treated without proper preliminary preparation will be considerably higher and the same degree of smoothness will perhaps never be attained. The engineers estimate that a two-man patrol, with a suitable truck and the necessary premixed material, can patch all breaks in 40 to 60 miles of highway. They say that

it is extremely important that such patching be kept up continuously and that small breaks be repaired immediately. Otherwise moisture collects in the depressions, saturates the surface under the adjacent oil top, and the break spreads with great rapidity. Experience up to this time indicates that the cost of patching and other maintenance will be less than the cost of the dragging and blading of the untreated sections.

The character of the oil used in the work done in Division I varied widely. That used in Humboldt County had a specific viscosity (Engler) of about 10 at 122° F. The asphalt content (80 penetration) ranged from 60 to 65 per cent. This oil is identical with most of the oil used in the Oregon work, and an inspection of the treated sections about March 1, 1917, indicated that it is too light for use in localities in which the rainfall is as heavy as in the California county. The specific gravity of such oil is about 0.94. If, therefore, rainfall is heavy and the road metal is moved or disturbed, the oil floats off on the surface of the water. This condition was particularly noticeable on the sections above described which had been covered with a rather thick layer of mulching. The rainfall for the season at the time of the inspection had been about 50 inches, and it is estimated that in places one-half of the oil had been removed in the manner described. Where the road metal had been thoroughly compacted prior to oiling the oil had withstood the winter rains quite satisfactorily.

On other parts of the same highway in Mendocino County the oil used had a specific viscosity of 45 to 80 at 122° F. This oil has a considerably higher specific gravity, and the surfaces treated with it were generally tight and uniform and showed no evidence of appreciable loss.

Experience of maintenance engineers verified by measurements justifies the estimate that the average loss of metal on the Redwood Highway was about 300 tons per mile per year, and the cost of replacing this metal would average not less than \$750 per mile. The average cost of oil treatment in Division I last year was about \$500 per mile, including such screenings as were necessary to produce a covering. Table 8 shows the cost of treating sections in the same division where little or no screenings were added.

TABLE 8.—Cost of surface treatment in Division I, California, 1926 (practically no cover material used)

Project	Length	Width	Quantity of oil	Cost per mile				
				Labor	Rental	Oil ¹	Miscellaneous	Total
	Miles	Feet	Gals. per sq. yd.					
Men.-1-A.....	0.73	18	0.45	\$98	\$78	\$233	\$27	\$436
	1.23	18	.47	192	115	215	29	551
Men.-1-B.....	4.12	18	.50	109	81	291	24	505
	4.11	18	.44	71	58	236	18	383
	1.4	18	.45	167	71	235	29	502
Men.-1-D.....	1.11	18	.30	91	54	158	18	321
	3.96	18	.38	90	33	198	14	335
Men.-15-A.....	13.1	18	.30	50	33	154	7	244
Men.-70-A.....	1.4	18	.46	84	70	239	14	407
	2.2	18	.33	114	43	156	11	324
Men.-1-E ₂	2.61	18	.28	98	68	199	30	395
Lak.-16-A.....	1.15	21	.49	176	185	301	82	744
Son.-1-A.....	4.3	18	.54	163	92	242	35	532
Total and average.....	41.42							379

¹ Cost of oil at refinery, \$1.04 per barrel. Freight averaged substantially the same amount. Average cost of oil, including freight, 4.9 cents per gallon.

The oil treatment as here applied was effective in stopping loss of metal and would be economical if the operation had to be repeated annually. That it will not have to be charged off annually but that it may be expected to give at least two or three years of average service is evidenced by the present condition of work done last year, at least 80 per cent of which is in first-class condition.

The cost of oil treatments in other counties and divisions of California is shown in Table 9. Tests of the oils used in these treatments are reported in Table 10.

TEMPORARY OIL TREATMENT UNDER HEAVY TRAFFIC IN SHASTA COUNTY

Similar oil treatments have been carried on in several other divisions of California but not so extensively. Two other sections of surface treatment show the pos-

sibilities of the use of such treatment for temporary purposes under heavy traffic. One of these is a section north of Redding designated as II-Sha.-3-B. This section, 6.12 miles in length, has an average traffic of about 1,500 vehicles per day, and an occasional maximum of 3,000 or more during midsummer. Crushed rock surfacing 21 feet in width had been completed in 1925 and maintained by dragging and blading. Considerable binder material had been added and the road was thoroughly compacted, and calcium chloride had been used as a dust palliative during the 1925 season.

This section was surface treated in June, 1926, the general method being similar to that followed in Oregon and Division I in California except that sweeping was continued much longer than is the usual practice. The road was swept and reswept until the rock particles were exposed to a depth of one-fourth to one-half

TABLE 9.—Cost of surface treatment in California, 1926 (with cover material)

Project	Length	Width	Quantity of oil	Operation	Cost per mile ¹				
					Labor	Equip-ment	Material	Miscellaneous	Total
I Hum.-1-AB	Miles 4.3	Feet 16	Gals per sq. yd. 0.5	Preparing base Oiling Cover Total	 \$89 105 194	 89 184 218	 \$5 132 130 267	 \$4 27 8 39	 \$18 432 268 718
I Hum.-1-C	13.9	16	.56	Preparing base Oiling Cover Total	 3 97 100	 13 121 134	 170 214 384	 1 49 50	 17 437 668
Hum.-1-D	11.2	16	.64	Preparing base Oiling Cover Total	 6 79 85	 28 112 140	 196 275 471	 2 41 43	 36 428 739
II Sha.-3-B	6.12	21	.5	Preparing base Oiling Cover Traffic protection Total	 128 61 138 327	 152 51 36 239	 257 153 410	 38 11 10 96	 318 380 337 1,072
II Sis.-3-B	18.9	24		Preparing base Oiling Cover Traffic protection Miscellaneous Total	 120 81 145 346	 85 63 77 225	 46 317 38 401	 58 33 25 412	 309 494 285 1,384
III Gle.-45-A	2.0	18		Preparing base Oiling Cover Miscellaneous Total	 13 44 21 78	 8 20 5 33	 188 62 250	 10 10 10	 21 252 88 371
III Col.-15-E	2.0	18		Preparing base Oiling Cover Miscellaneous Total	 6 64 52 122	 25 43 30 98	 181 218 399	 27 27 27	 31 288 300 646
III Ama	6.5	19	.52	Preparing base Oiling Cover Miscellaneous Total	 65 43 117 225	 25 48 83 156	 18 181 26 225	 40 57 97	 108 312 226 703
IV S. F.-55-A; S. M.-55-A ₁	5.5	33	.5	Preparing base Oiling Cover Miscellaneous Total	 63 96 188 347	 35 45 161 241	 328 276 604	 9 14 13 128	 107 483 638 1,320
IV Mrn.-52-A	4.89	18	.4	Preparing base Oiling Cover Total	 64 31 50 145	 53 21 57 131	 154 116 270	 3 18 14 35	 120 224 237 581

¹ Average cost per mile reduced to 18-foot width, \$830.

² Mostly patching.

TABLE 10.—*Typical California fuel oils used in bituminous treatment by the California Highway Commission in 1926*

Type of treatment and project	Lab. No.	Date sampled	Specific gravity	Baumé gravity	Specific viscosity, Engler, 122° F.	Viscosity, Saybolt-Furol, 122° F.	Per cent of asphalt, 80-penetration
Surface treatment							
I-Hum.-I-C.	1540	June 23	0.9409	18.8	9.1	—	51.0
I-Hum.-I-D.	1541	July 10	0.9409	18.8	9.1	—	52.9
I-Hum.-I-G ₁	1741	Oct. 8	0.9453	18.1	10.9	—	55.0
I-Hum.-I-G ₂	1742	do.	0.9416	18.2	10.5	—	55.5
I-Hum.-I-G ₃	1683	Sept. 11	0.9557	16.5	10.8	—	54.4
I-Hum.-I-I ₁	1690	Sept. 4	0.9743	13.7	126.7	—	73.8
I-Men.-I	1446	May 25	0.9742	13.7	68.1	—	74.5
I-Men.-I	1458	May 26	0.9642	15.2	73.8	—	81.2
I-Men.-I	1479	June 9	0.9655	15.0	76.4	—	52.5
I-Men.-I	1480	do.	0.9695	14.4	83.7	—	77.8
I-Men.-I	1526	June 25	0.9658	14.8	64.4	—	71.0
I-Men.-I	1528	do.	0.9722	14.0	71.1	—	69.1
II-Sha.-3B ₁	1469	May 25	0.9675	14.7	44.8	—	62.0
II-Sha.-3B ₂	1470	May 29	0.9688	14.8	52.3	—	66.4
II-Sha.-3B ₃	1471	June 2	—	—	43.7	—	63.2
II-Sha.-3C	1487	June 12	0.9668	—	59.0	—	57.0
II-Sis.-3B ₁	1713	Sept. 16	0.9749	13.6	68.0	—	60.0
II-Sis.-3B ₂	1714	Sept. 18	0.9749	13.6	68.0	—	63.0
II-Sis.-3C	1672	Sept. 2	0.9695	14.4	54.6	—	63.0
II-Sis.-3C	1709	Sept. 6	0.9688	14.5	48.5	—	65.8
II-Sis.-3C	1711	do.	0.9722	14.0	50.4	—	64.8
II-Sis.-3C	1712	do.	0.9681	14.6	50.0	—	66.5
Surface mixing method							
VIII-Imp.-26G	1654	Aug. 25	0.9550	16.6	23.9	117	67.4
VIII-Imp.-26G	1655	Aug. 26	0.9537	16.8	25.9	115	64.5
VIII-Imp.-26G	1656	do.	0.9531	16.9	24.4	116	64.1
VIII-Imp.-26G	1657	Aug. 27	0.9557	16.5	35.8	115	69.9
VIII-Imp.-26G	1658	do.	0.9544	16.7	30.1	115	64.5
VIII-Imp.-26G	1659	Aug. 28	0.9492	17.5	29.8	115	63.6
VIII-Imp.-26G	1663	Aug. 20	0.9655	15.0	30.9	—	64.6
VIII-Imp.-26G	1664	Aug. 30	0.9655	15.0	28.2	—	63.8
VIII-Imp.-26G	1665	Aug. 31	0.9531	16.9	24.8	116	65.6
VIII-Imp.-27B ₁	1433	May 1	0.9428	18.5	16.9	—	63.0
VIII-Imp.-27B ₂	1431	Apr. 30	0.9459	18.1	18.4	—	67.8
VIII-S. Bd.-31D	1772	Aug. 24	0.9628	15.4	30.9	—	62.7
VIII-S. Bd.-31D	1773	Aug. 25	0.9681	14.6	49.4	215	62.3
VIII-S. Bd.-31D	1775	Aug. 26	0.9688	14.5	50.0	215	67.4
VIII-S. Bd.-31D	1776	Aug. 27	0.9628	15.4	26.7	—	63.4
VIII-S. Bd.-31D	1777	Aug. 30	0.9662	14.9	26.4	—	63.3
VIII-S. Bd.-31-Skr	1778	do.	0.9675	14.7	52.0	184	69.7

inch. This was to insure a nonskid texture on account of steep grades and sharp curves. Fuel oil having an asphalt content of about 65 per cent was then applied in two applications of one-fourth gallon each. The oil had a specific viscosity (Engler at 122° F.) of 45 to 50. Because of its high viscosity, the oil penetrated slowly and chips were added after each application. The chips ranged in size from three-fourths inch to 10-mesh, and the amount used averaged about 50 cubic yards per mile. The heavy, sticky oil effectively held the chips in place, and these, together with thorough sweeping prior to treatment, gave the resulting surface the desired nonskid texture closely resembling that of bituminous macadam, as shown in Figure 27. This texture remained unaltered under traffic throughout the season.

Roughometer readings⁶ taken on this section during the latter part of August, 1926, showed a roughness of 25 to 30 inches per mile, which corresponds with that usually found on first-class bituminous macadam.

The cost of treating this section was \$1,072 per mile, or 8.7 cents per square yard, compared with \$726 per mile of 16-foot width, or 7.7 cents per square yard for similar work in Division I. The difference lies in the thorough preparatory sweeping and in the amount of chips applied. Table 9 shows detailed costs of surface treatment on this and other sections upon which cover material was used in considerable quantities.

H. S. Comly, division engineer in charge of this construction, states that the average cost of maintenance

on a 60-mile section of untreated road between Redding and Dunsmuir, including the section above described has been averaging about \$1,900 per mile per year under traffic of about 1,500 vehicles. This cost of maintenance included the cost of such metal as was replaced, calcium chloride dust palliative, and sprinkling. The experience already indicates considerable reduction as a result of the oil treatment. In addition, a smoother, nonskid, dustless surface has an increased service value difficult to estimate.

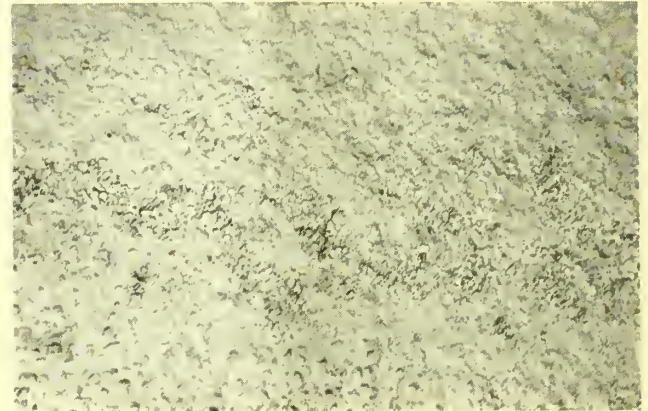


FIG. 27.—PICTURE TAKEN ON THE PACIFIC HIGHWAY NEAR REDDING ONE MONTH AFTER COMPLETION OF SURFACE, SHOWING SURFACE TEXTURE WHICH WAS RETAINED UNDER HEAVY TRAFFIC

THE SKYLINE BOULEVARD

Another section, 5½ miles long, was treated during the season of 1926 in Division IV near San Francisco under traffic even more severe than those above described. The section is a pleasure drive extending southerly from the city of San Francisco and carries little truck traffic.

Traffic, according to the 1926 census, averages about 2,000 vehicles per day with an occasional Sunday maximum of several times this number. The road had been surfaced with crushed rock, the northerly 3½ miles to a width of 35 feet and the remainder 29 feet. Light oil had been used as dust palliative but was relatively ineffective under this extremely heavy traffic. Patches of oil-bound surface had formed which tended to add to the roughness of the road, but it is probable that the use of the light oil had somewhat reduced the loss of the metal. It was not otherwise successful. The section was given a surface treatment in August, 1926, under the supervision of John H. Skeggs, division engineer. Preparation for the new treatment consisted of smoothing, blading, and thorough sweeping. The mulching on the road was hand screened to remove the dust and the coarser material was used to cover the fresh oil which was spread in two applications of approximately one-fourth gallon each. Additional screenings were applied as needed, and on the completion of oiling a program of intensive patching and mending was undertaken. Depressions were carefully filled with pre-mixed material and compacted under traffic. The construction and maintenance produced a very satisfactory surface which has withstood traffic for about a year. Roughometer readings on February 17, 1927, gave an average roughness of 25 to 30 inches per mile, which, as has been stated, is about normal for first-class bituminous macadam. A section of asphaltic concrete adjoining this highway within the city limits

⁶ Readings were made with a Bureau of Public Roads roughometer operated at 20 miles per hour and calibrated to give results comparable with those obtained on the same road with a violog. The calibration consisted of running the car over five 1 by 10 inch boards placed at 50-foot intervals on a smooth surface. The result is taken to represent "10 inches" of roughness and a conversion factor is established.

of San Francisco gave a roughness of 40 inches per mile.

The average cost of the treatment on the Skyline Boulevard was \$1,320 per mile and the width of the surface treated averaged about 33 feet. The cost, width being considered, was about average.

With the cooperation of the automobile clubs, traffic was kept off the oil during the treatment and work was carried on without complaint from the traveling public.

GENERAL COMMENTS ON THE METHOD OF SURFACE TREATMENT

On the basis of the cooperative study to date and the inspection of the methods and results of surface oiling in Oregon and California, several general comments are pertinent.

It is apparent that the cost of this bituminous treatment in many cases is less than the value of the metal which would otherwise be lost in a single year.

The treatment, where properly conducted results in a dustless surface which is much smoother than the untreated crushed rock and gravel surface; and experiments by the Washington State College indicate that the tire wear caused by roads of this type is no greater than that caused by more expensive pavements.

Two physical features of the road to be treated are most important. There should be an adequate thickness of surfacing, and the binder in the road metal should not be a heavy, unstable clay. Such binder has given satisfactory results in some cases in untreated roads, but the application of oil forms a waterproof cover which holds moisture in the road, and clays of this character remain saturated, unstable, and without cementing value.

It has been demonstrated by experience that the treatment can be employed successfully under climatic conditions as widely varying as those of eastern Oregon and Humboldt and Mendocino Counties in California. In eastern Oregon the roads have been subjected to a relatively low annual precipitation, to long hot summers, and winters in which the temperature goes well below zero. It has been demonstrated in this section that satisfactory results can be obtained where extremes of temperature prevail except where the surface metal is mixed with unsatisfactory clay binders. The sections in Humboldt and Mendocino Counties show that a successful result can be obtained under conditions of long-continued rainfall; but in such wet climates it has been found that variations in the characteristics of the oil have far more influence than is apparent in the drier, colder sections.

In the drier climates, where considerable penetration is desired, the light grade of "fuel oil" has given excellent results. In sections where rainfall is heavy, it has been found that the heavier "fuel oil" should be used, at least in the surface coat. Heavy oil, however, requires more screenings than light oil and is, therefore, more costly. In certain localities a first application of light oil followed by an application of heavy oil will accomplish the desired results and keep the cost of screenings at a minimum. It is apparent, therefore, that the selection of the bituminous material requires careful consideration.

A nonskid texture can be secured on a road which is bound with stone dust or inert binder by sweeping before treatment until the stone particles are exposed. A covering of screenings on the last coat of oil improves the texture.

Intelligent and efficient maintenance beginning immediately after construction is essential to the success of surface treatment, and if a highway organization is not in a position to carry on such maintenance work the treatments herein described are not recommended.

Accurate cost data as to maintenance of crushed rock and gravel roads thus treated with light "fuel oil" are not yet available, but indications point to a cost of \$300 per mile per year with traffic of about 500 vehicles.

Freshly oiled surface is readily damaged by iron tires which pick up the oil and leave bare tracks that do not entirely heal. Rubber tires do not appreciably damage such a road; but a reckless driver on a freshly oiled surface not only endangers himself and his own property but is a menace to others. Skidding upon the recently oiled roadway has caused several accidents. Also it is very difficult to drive a car over such a surface without spattering it from end to end with oil. The cleaning of a car in this condition costs from \$5 to \$10, and a considerable loss in the aggregate may result to motorists on a road carrying heavy traffic. One thousand cars per day would mean a cleaning charge of \$5,000 per day or more. If this damage has resulted from travel over 5 miles of freshly oiled surface, the charge is \$1,000 per mile per day against the oil treatment. This damage might accumulate for several days and would largely nullify the benefits of this very economical type of surface treatment. Traffic protection should receive most careful attention, otherwise the treatment is likely to fall into disfavor. Especially would this be the case if oiling proceeds slowly, new sections being opened as others are finished.

The application of chips in the amount of about 40 or 50 cubic yards per mile (10 or 12 pounds per square yard) immediately after oiling reduces the damage to rolling stock. This is not sufficient, however, unless cars are operated at very slow speed. Where detours are available, the road should be closed to traffic until the oil has penetrated. Chips may be applied, but rather to add nonskid quality to the road surface than as a protective measure.

If traffic can not be detoured and the road closed during treatment, oiling should proceed in successive strips, leaving enough untreated or completely covered roadway to care for traffic. In any event, traffic should be cautioned that a speed in excess of 8 miles per hour on a freshly oiled surface, even for a few hundred feet, will result in an expensive cleaning job.

Where traffic is carried on a road under treatment, traffic control is ordinarily installed. The usual control is not sufficiently rigid to provide protection. Such a control system, it is suggested, should consist of special officers stationed at each end of the freshly oiled road. The officers should stop each car and present the driver with a ticket showing the time at which the car will reach the other end of the control if operated at a speed of not more than 8 miles per hour. The driver should be told that he will not be permitted to pass the control at the other end until the time indicated. Merely cautioning the drivers is not sufficient, as has been demonstrated in many cases.

Too much stress can not be placed on the importance of such protective measures. The motorists will benefit after oil treatment by the complete elimination of dust, but this subsequent benefit is forgotten when an owner discovers that his car is rather thoroughly covered with black, sticky oil.

BITUMINOUS TREATMENT OF CRUSHED ROCK AND GRAVEL ROADS BY THE SURFACE MIXING PROCESS

Wisconsin ⁷ began using a surface-mixing process in 1923, Wyoming experimentally in 1925, and California in 1926. The immediate occasion in each case was a section of road with too much loose material on the surface to permit use of the ordinary surface-treatment process; but promise of durability, extraordinary riding qualities, and special adaptability to fine crushed surfaces indicate a wide field for the process.

Wisconsin reports a cost of about \$1,250 and California an average of about \$1,268 a mile for an 18-foot width of this type of treatment. (See Table 11.) The lower cost of oil in California as compared with tar in Wisconsin permits a greater thickness of treated material for about the same cost. No figures are available as to the cost of maintenance, but both States are convinced that it will be less than the maintenance of surface-treated roads.

TABLE 11.—Cost of surface-mixing treatment on southern California projects, 1926

Project	Length Width		Annual cost of untreated surface per mile			Quantity of oil applied	Cost of surface-mixing treatment per mile				
			Ordinary maintenance	Loss of metal	Total		Oil	Labor	Equipment	Supplies and miscellaneous	Total
	Miles	Feet				Gals. per sq. yd.					
Imp. 26-G	8.81	20	\$3,310	\$3,097	\$6,407	1.5	\$893	\$168	\$260	\$83	\$1,404
Imp. 26-A	3.93	20	1,342	1,890	3,232	1.5	1,017	118	193	35	1,363
Imp. 27-B	14.07	20	180	1,010	1,190	1.5	783	282	268	97	1,430
S. Bd. 31-D	13.60	18	2,640	1,958	4,598	1.25	535	272	290	61	1,158
S. Bd. 31-E	4.60	18	2,372	4,444	6,816	1.25	697	298	339	246	1,580
Average per mile.					3,980						1,315

Reduced to 18-foot road, the average cost of the mixed surface treatment is \$1,268 per mile.

Experience with the surface-mixing method is too short to permit an accurate estimate of probable life. However, examination of a number of treated earth roads constructed 10 or more years ago shows no deterioration of the oil binder with age, and, apparently, the oil treatment should have the life of the underlying structure, which depends upon such factors as weight and volume of traffic, adequate thickness, subsoil, and climate. The residual asphaltic oil does not evaporate and hardens but slowly. Roughness and breaks in the surface are to be remedied by reworking or patching as a part of routine maintenance.

Although construction costs are relatively low, discussion of the economics of the type requires that a term be assumed within which the cost of treatment should be written off. Depending as it does upon many factors, the term must be rather arbitrarily selected. It is suggested that the period be assumed to lie between 5 and 10 years, dependent upon subsoil, climate, and probable increase of traffic. The type is believed to be adequate for a maximum of 1,000 vehicles per day. If the treatment is an emergency measure to protect a road of insufficient thickness, the outcome is too uncertain to justify assuming life beyond the current maintenance year.

This process presumes prior construction of a roadway of strength ample to carry the loads, which implies a sufficient thickness of surface material and also compactness beneath the oil mixture. A layer of fine, loose material between the compacted base and the treated surface should be avoided. Mechanical operations will be difficult where the road metal contains any considerable quantity of large stone, but are simple on fine crushed rock or gravel. The presence of heavy clay binder is objectionable, but the proper amount of fine inert filler is an advantage.

The viscosity of the oil used is an important factor in this process. It must not be too heavy to mix with the road metal readily at prevailing temperature. Quantity is even more critical; an excess results in rutting and other displacement; a deficiency causes raveling. Heretofore the only gauge has been the appearance of the mixture, but the similarity to other asphaltic types and the expense and unreliability of cut-and-try methods suggest the possibility of utilizing a modified form of the stain test, once common in sheet asphalt practice. As stated elsewhere in this report, experiments along this line have been very promising. Uniformity is difficult to attain because inevitably the thickness will vary from place to place, and with uniform application of oil the result will be an irregular richness. If the irregularity is sufficient to be observed during the construction process, additional fine material beneath the richer portions should be loosened and incorporated into the oiled material.

THE SURFACE-MIXING METHOD AS USED IN CALIFORNIA

So far as known no metaled roads on the Pacific coast were treated according to the Wisconsin method or in a similar manner until the summer of 1926. During the latter part of that year approximately 50 miles of this type was constructed on the California highway system in San Bernardino and Imperial Counties with, however, a number of differences in the details of the process. Eighteen miles of this work is in San Bernardino County between Victorville and Barstow, and the remainder in Imperial County between Westmoreland and Yuma, Ariz. This is a desert region where the rainfall is very light, the summers long and extremely hot, and consolidation for surface treatment is impractical. The application of the surface-mixing method to graveled roads in this area was developed by E. Q. Sullivan, division engineer, California Division of Highways, and his assistant, J. E. Stanton.

Experimental section.—An experimental surface-mixed section 700 feet in length was constructed near Westmoreland, Imperial County, in June, 1926. The road had been surfaced with crushed gravel to a width of 18 feet in May, 1924 (contract No. 430, Imp.-26-A), but had never compacted under traffic. This first experimental treatment consisted of the application of 1.75 gallons per square yard of "fuel oil" containing approximately 65 per cent of 80-penetration asphalt. The oil was spread to a width of 15 feet and mixed with the loose gravel by blading and working until the mixture was more or less uniform in color. When inspected on July 7, 1926, the oil mixture appeared richer on the northerly side of the road, in some places contained more sand than in others, and in general

⁷ Tar Surface Treatment of Gravel Roads, by N. M. Isabella, PUBLIC ROADS, vol. 6, No. 2, April, 1925, p. 40.

lacked uniformity. However, it was reasonably smooth and did not displace readily. The thickness of the mixture was found to be 2 to 2½ inches. The results of this experiment were encouraging and similar treatment of the 18 miles of the San Bernardino-Needles route was undertaken.

San Bernardino-Needles route.—The sections treated on this highway consisted of 13.6 miles from Victorville northerly and 4.6 miles between Hicks and Barstow. These sections of highway were surfaced in the early part of 1926 with 4 to 6 inches of gravel crushed to pass a 1-inch circular opening. (Contracts 476 and 489, S. Bd.-31-D-E-F.) The subgrade consisted of disintegrated granite and gravelly desert soil. Because of heat and lack of rainfall, the surfacing material did not bind under traffic, and addition of disintegrated granite and other binders failed to remedy conditions, so that, in July, 1926, there was a blanket of 1 to 2 inches of fine, loose gravel covering the entire roadway. The road was dusty and much metal was being lost. The effort to bind the road metal is reflected in the high maintenance costs shown in Tables 1 and 11.

The treatment adopted was based upon the experimental section just completed near Westmoreland and was as follows:

1. The road metal was scarified to a uniform depth of about 2 inches.

2. Oil was spread upon one-half of the road in three approximately equal applications to a total quantity of 1.25 gallons per square yard. After each application the material was partially mixed with disk cultivators and spring-tooth harrows sufficiently to remove the excessive stickiness of clear oil. The other half of the roadway was then treated in like manner. When thus partially mixed the roadway was passable for traffic without damage to vehicles or to the road, which was a material convenience where detours were not available. The oil had an asphaltic content of 60 to 70 per cent and specific viscosity (Engler) ranging from 25 to 101 at 122° F.

3. The partially mixed material was gathered into windrows by blade graders, and these windrows were moved from side to side of the road until the material was thoroughly mixed as indicated by uniform color.

4. The mixed material was spread by blade graders to the final cross section and compacted by traffic. A blade grader was continued in operation for several days during the compacting process to fill ruts and remove irregularities.

On the Victorville section the treated material averaged 2.4 inches in thickness at the center and 1.74 inches at the sides after being compacted. On the Hicks-Barstow section the thicknesses averaged 3.04 and 1.43 inches, respectively.

The cost of the treatment ranged from \$1,158 to \$1,580 per mile for the width of 18 feet. The details are shown in Table 11.

A significant and even startling loss of road metal occurred during the six months interval between the completion of gravel surfacing and the treatment with oil. (See Table 1.) On the Victorville section measurements showed a reduction of the average thickness from 5 to 4.22 inches. At \$2.79 per ton the value of the lost metal was \$979 per mile in the period of six months. The Hicks-Barstow section suffered a reduction of average thickness from 5 to 3.23 inches, which at the same unit price amounts to \$2,222 per mile for the six-month period. General failure of the treated road due to lack of metal would have been probable

in the near future if the subgrade had not been mainly disintegrated granite or stable sand-clay.

The surface of the treated road is clean, dustless, reasonably nonskid, and smooth. Roughometer readings taken in October, 1926, immediately after completion showed a total roughness of only 10 inches per mile. In July, 1927, readings were again taken and the roughness was approximately the same. For comparison, it should be stated that the usual degree of roughness on first-class asphaltic pavement ranges from 15 to 25 inches per mile and on best concrete from 6 to 10 inches. In so far as smoothness is concerned, the mixing method as practiced on this section gave better results nine months after completion than hot-mixed asphaltic concrete. The reason for the smoothness is the method of manipulation, by which the bituminous mixture is compacted gradually under traffic from the base upward rather than downward from the top as occurs under a roller in ordinary bituminous construction. The use of long-wheel-base blades removes corrugations or other irregularities which might otherwise form during construction and the low bituminous content probably contributes to stability.

For comparison with the treated sections, roughness readings were taken on an untreated section 7 miles in length between the two treated sections. These averaged 119 inches per mile, showing that the untreated surface was twelve times as rough as the treated sections. The untreated gravel was maintained in the usual manner by blading and keeping a loose mulch over the surface and, in addition to being much rougher than the treated surface, was very dusty and was probably being worn away at such a rate that the loss would equal the cost of oiling in a few months.

SURFACE MIXING IN IMPERIAL COUNTY

Mixed bituminous treatment was applied also to 27.4 miles in the Imperial Valley in September and October, 1926. These sections had been surfaced with crushed gravel (contracts 430, 438, and 465) in 1924 and 1925, but, excepting the Sand Hills-Yuma portion, had never properly compacted because of extreme heat and lack of humidity. During subsequent maintenance disintegrated granite binder was added between El Centro and Imperial and water was freely used but without accomplishing much toward compacting the material or reducing the dust nuisance. In 1925 several applications of 40 to 50 per cent asphaltic oil was spread at the rate of one-seventh to one-tenth gallon per square yard in a further effort to overcome the dust, but the looseness of the surfacing material rendered this ineffective except on the Sand Hills-Yuma section, where some improvement resulted. It afterwards appeared that the oil used on this portion of the road actually contained as much as 68 per cent of asphalt, but the amount applied was not sufficient to produce a permanent wearing surface.

The mixing method finally adopted on these Imperial Valley sections was quite similar to that used in San Bernardino County, excepting that the quantity of oil was at first reduced to 1 gallon per square yard. After completing the treatment with this amount of oil evidence of insufficiency developed. The treated material was again scarified and an additional one-half gallon of oil per square yard was applied, the mixing process resumed, and finally the mixed material was distributed with a blade grader and compacted



A



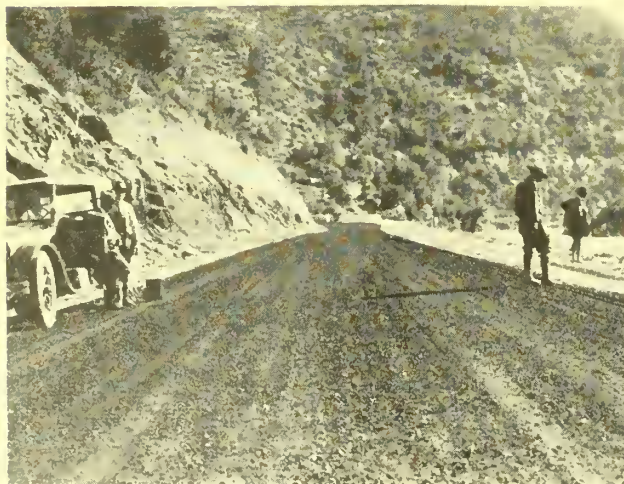
B



C



D



E



F

FIG. 28.—METHOD OF APPLYING OIL TREATMENT

- A.— Disk and spring-tooth harrows following oil distributor
 B.— Oil applied to one side of road at a rate of 1.5 gallons per square yard and harrowing completed
 C.— One side of road stripped of oiled material, which is piled in a windrow just beyond the center
 D.— Grader mixing material by taking it from windrow and casting into another on far side of road
 E.— Condition of road surface on the day following final spreading with a 12-foot blade grader
 F.— Traffic compacted surface about one month after final spreading

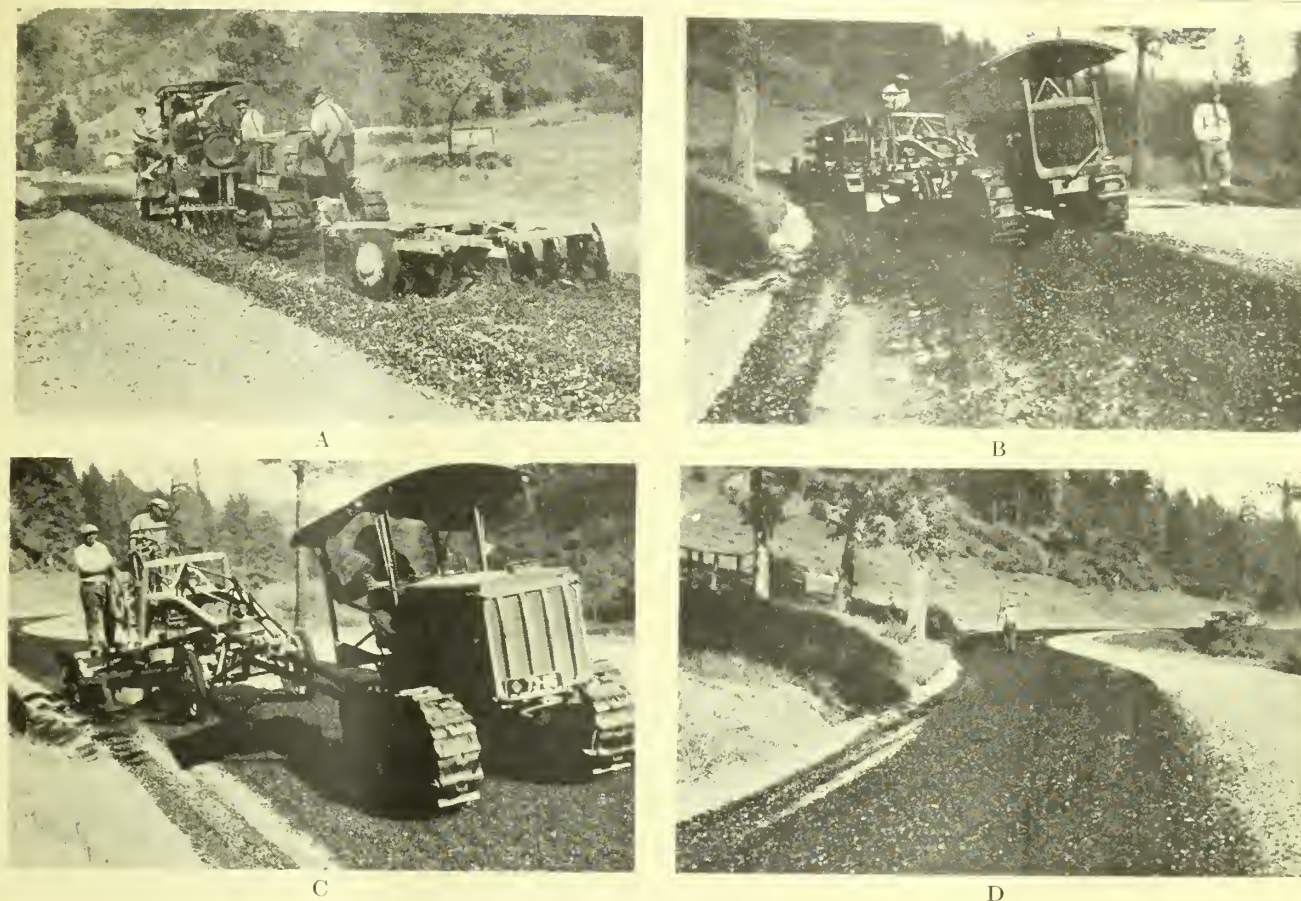


FIG. 29.—MODIFICATION OF THE MIXING PROCESS ON THE MARIPOSA-BRICEBURG ROAD, TREATING HALF THE ROADWAY AT A TIME

- A.—Third oil application followed by diskings. The material is somewhat coarser than on other projects
- B.—Manipulating material with tractor-drawn grader
- C.—Final distribution of mixed material
- D.—Half of roadway completed and ready for traffic

as in San Bernardino County. According to measurements made in February, 1927, the average thickness is about $2\frac{1}{2}$ inches.

The cost of the work in Imperial County was approximately \$1,400 per mile for 20 feet of width. The details are shown in Table 11.

The Imperial Valley sections, when measured for roughness in February, 1927, were found to be even smoother than the Victorville-Barstow Road in San Bernardino County.

The loss of metal on these Imperial Valley sections prior to oiling was very high. (Table 1.) On the Sand Hills-Yuma section, the loss amounted to \$1,010 per mile per year. On the others it ranged from \$1,726 to \$3,097 per mile per year. Consequently the material remaining was inadequate for oiling. In many places there was no metal between the oiled wearing surface and the clay and silt subgrade. Fortunately, however, the subgrade did not become saturated during the winter, and the road was in remarkably good condition in the spring.

On the section between Brawley and Imperial there are a number of places where the clay subgrade has become saturated and the surface rutted. A sample of clay from the subgrade beneath a rut had a lineal shrinkage of 7.7 per cent and a field moisture equivalent of 28.4 per cent, showing it to be unstable and unsuitable for subgrade purposes. The sections showing distress probably represent less than $1\frac{1}{2}$ miles out of the

total 9-mile length, and the defects do not interfere with the comfortable use of the road. If this section of highway had not been oiled, it could not have carried traffic through the winter of 1926-27. As a result of the expenditure of approximately \$1,400 per mile for oil treatment, 80 to 90 per cent of the surface has been saved from destruction, and after five months' use is, from the motorists' standpoint, equal to the highest type of pavement. The cost of oiling was less than the combined cost of maintenance and loss of metal would have been during the same five months under normal conditions. In view of the inadequacy of the metal on the road at time of treatment, it is not to be expected that this section can meet the heavy traffic requirements for an indefinite period without some reconstruction, but from the appearance at the present time it is believed that reconditioning of one-quarter of the mileage will care for the weak places and the remainder will give service for a considerable period.

The series of photographs in Figure 28 illustrate the various steps in the surface-mixing process as usually practiced; those in Figure 29 illustrate a modification of the process employed on the Mariposa-Briceburg Road, where half of the roadway was treated separately.

TESTS OF SURFACE MIXTURES

Analyses of samples taken from surface-mixed roadways show that apparently the amount of "fuel oil" required in a surface mixture is about three-fourths

the quantity of asphalt that would be used for the same grading. So long as the correct amount of oil is used, it appears that wide variations in grading may be tolerated. Samples were taken from successful bituminous surfaces produced by the mixing method where the material varied from silt to crushed gravel.

An empirical formula has been developed which is tentatively submitted with the thought that it may be of value in estimating the amount of oil required. It is not applicable to porous or absorbent materials such as cinders or lava but otherwise it appears to give results which are consistent with service obtained from various mixes. The formula is:

$$P = .015a + .03b + .17c$$

in which P is the percentage of oil required, a is the percentage of metal retained on a 10-mesh sieve, b is the percentage of metal passing the 10-mesh and retained on the 200-mesh sieve, and c is the percentage passing the 200-mesh sieve.

The stain test hereafter described, apparently furnishes the only reliable means for determining the amount of oil required for absorbent materials.

Used with nonabsorbent materials, the proportion of oil calculated from the formula has furnished satisfactory stains from many mixtures prepared in the laboratory; but even with such materials the indications of the stain test seem to be a more reliable guide than the formula in case of conflict between the two methods.

Stain test.—Inasmuch as the success of the surface-mixing method depends upon correct gauging of oil requirements and upon uniform mixing, an attempt was made to find a field test which would be of value to those carrying on such work. Resort was had to a modified form of the pat stain test used for many years in the control of sheet asphalt mixtures. This stain test can not be made on samples containing coarse sand or gravel. It is, therefore, necessary to prepare samples for test by removing coarse particles, and it is also necessary that the samples be heated. The stain test as finally developed is described as follows:

The sample from the roadway is first warmed. The fine material is then separated from the coarse by passing through a 10-mesh sieve. This can be readily accomplished by rubbing gently with the fingers and by loosening the fine particles that adhere to the coarse. The particles which do not pass the 10-mesh sieve may be discarded. The original sample should be of sufficient size to provide about 1 pound of material passing the 10-mesh sieve. This 10-mesh material is heated to approximately the boiling point of water, which may be conveniently accomplished by placing the sample in a fruit jar or can and allowing it to remain partially submerged in the boiling water for a period of about one hour. The heated mixture is then dumped in a pile on the center of a sheet of white typewriter paper and leveled to a thickness of about 1 inch, when another sheet of paper is placed on top. A wooden block 2 inches thick is placed on top of the paper and to this are delivered five blows from a 2-pound hammer, falling freely for a distance of about 1 foot. The two papers are then removed from the asphaltic mixtures and the stain produced indicates the relative amount of oil in the sample.

Satisfactory surface mixtures will produce a light yellowish brown stain, in which the impression of the individual sand particles may be distinguished and

which is not blurred or blotched. A heavy stain indicates the presence of excess oil, which is not only uneconomical but also causes displacement under traffic.

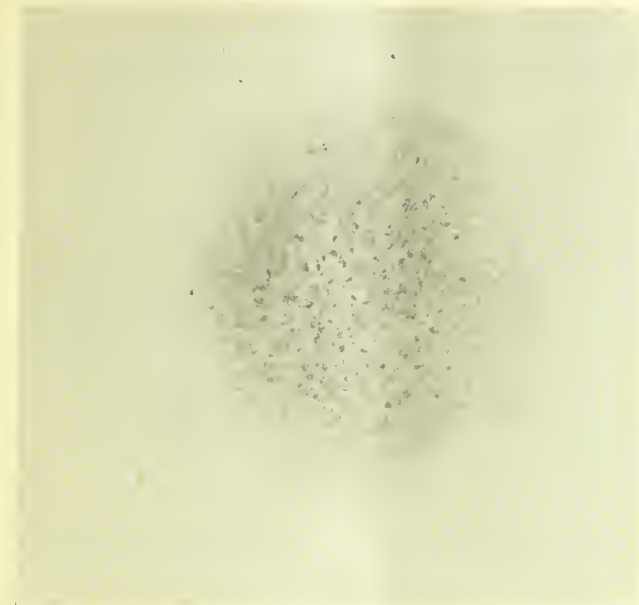
Samples of the completed surface mixture were taken from the sections of treated crushed gravel roads in San Bernardino and Imperial Counties, and also for comparative purposes from oiled earth and sand roads and shoulders in Imperial and Riverside Counties. These samples were subjected to mechanical analysis and to the above-described stain test with interesting and illuminating results. The results of typical tests are shown in Figures 30, 31, and 32.

The asphalt content of the samples taken from the crushed gravel roads in Imperial and San Bernardino Counties ranged from 4.3 to 5 per cent of the total mixture, or from 7.8 to 10 per cent of the sand present (material passing the 10-mesh sieve). The proportion of asphalt to fines in the treated earth and sand roads is also very similar. Examined in the light of the stain tests and the observed behavior, it is apparent that the amount of bitumen required in both the crushed gravel and earth mixtures varies with the grading of the material.

Tests 106, 110, 111, and 113, the results of which are shown in Figure 31, are particularly illuminating. Sample 106 was taken from an oiled earth road in Riverside County. The natural road material in this case was a fine silty soil and the treatment consisted of



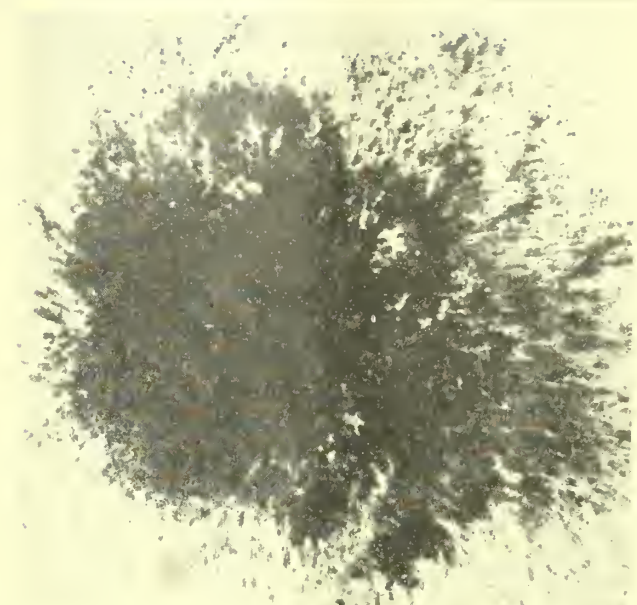
FIG. 30.—TYPICAL OIL STAIN



Test No. 110-VIII—Imp.-26-C. Crushed gravel treated by mixing method; bituminous surface, 2½ inches thick

Grading of total sample		Grading of material finer than 10-mesh	
Size	Per cent passing	Size	Per cent passing
200-mesh.....	11.0	200-mesh.....	24.0
10-mesh.....	45.8	100-mesh.....	38.2
3-mesh.....	82.4	50-mesh.....	50.7
½-inch.....	95.8	40-mesh.....	56.6
¾-inch.....	97.5	30-mesh.....	62.9
1-inch.....	100.0	20-mesh.....	74.5
		10-mesh.....	100.0

Bitumen: 4.6 per cent of total sample; 3.7 per cent theoretical requirement.



Test No. 111-VIII—Imp.-27-B. Oiled blow sand; shoulders adjacent to pavement soft and loose

Grading of total sample		Grading of material finer than 10-mesh	
Size	Per cent passing	Size	Per cent passing
200-mesh.....	4.6	200-mesh.....	4.6
10-mesh.....	97.1	100-mesh.....	24.0
3-mesh.....	98.0	50-mesh.....	50.2
½-inch.....	100.0	40-mesh.....	73.3
		30-mesh.....	88.2
		20-mesh.....	97.1
		10-mesh.....	100.0

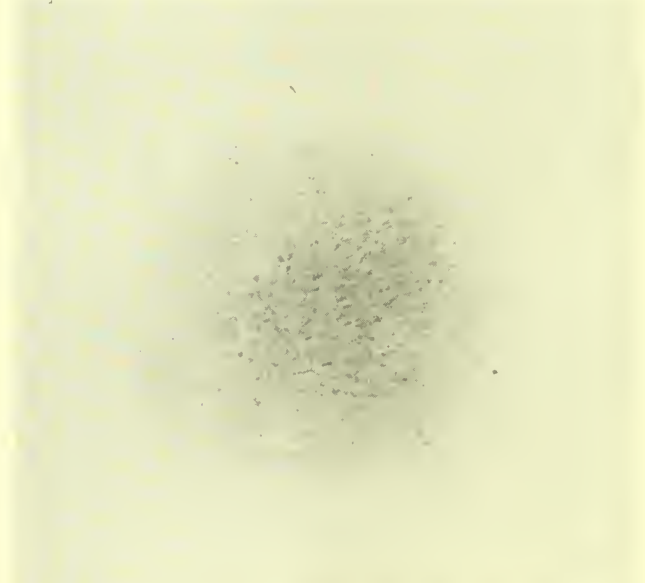
Bitumen: 12.4 per cent of total sample; 4.1 per cent theoretical requirement.



Test No. 113-VIII—Imp.-12-A. Sand-oil detour road; bituminous treatment, 7½ inches thick

Grading of total sample		Grading of material finer than 10-mesh	
Size	Per cent passing	Size	Per cent passing
200-mesh.....	4.1	200-mesh.....	4.9
10-mesh.....	79.7	100-mesh.....	8.6
3-mesh.....	97.9	50-mesh.....	18.2
½-inch.....	99.4	40-mesh.....	26.6
¾-inch.....	100.0	30-mesh.....	36.5
		20-mesh.....	61.0
		10-mesh.....	100.0

Bitumen: 3.4 per cent of total sample; 3.3 per cent theoretical requirement.



Test No. 106, Coachella Valley—Riverside County. Oiled dirt road (mixing method) treatment about 5 inches thick

Grading of total sample		Grading of material finer than 10-mesh	
Size	Per cent passing	Size	Per cent passing
200-mesh.....	43.8	200-mesh.....	43.8
10-mesh.....	99.8	100-mesh.....	55.6
3-mesh.....	100.0	50-mesh.....	82.2
		40-mesh.....	89.2
		30-mesh.....	93.3
		20-mesh.....	98.1
		10-mesh.....	100.0

Bitumen: 9.4 per cent of total sample; 9.2 per cent theoretical requirement.

FIG. 31.—TYPICAL OIL STAINS

Test No. 112-VIII-Imp.-27-B₂. Crushed gravel treated by mixing method; bituminous surface, 3-inch thick, is very hard and smooth

Grading of total sample		Grading of material finer than 10-mesh	
Size	Per cent passing	Size	Per cent passing
200-mesh.....	11.0	200-mesh.....	19.7
10-mesh.....	55.3	100-mesh.....	44.7
3-mesh.....	79.5	50-mesh.....	56.0
1½-inch.....	89.9	40-mesh.....	63.9
¾-inch.....	97.9	30-mesh.....	68.1
1-inch.....	100.0	20-mesh.....	78.2
		10-mesh.....	100.0

Bitumen: 4.3 per cent of total sample; 3.9 per cent theoretical requirement.

Test No. 108-VIII-Imp.-26-A. Crushed gravel treated by mixing method; bituminous surface, 2½ inches thick

Grading of total sample		Grading of material finer than 10-mesh	
Size	Per cent passing	Size	Per cent passing
200-mesh.....	12.9	200-mesh.....	22.6
10-mesh.....	58.5	100-mesh.....	41.4
3-mesh.....	75.9	50-mesh.....	57.7
1½-inch.....	85.5	40-mesh.....	64.0
¾-inch.....	96.5	30-mesh.....	70.0
1-inch.....	100.0	20-mesh.....	81.9
		10-mesh.....	100.0

Bitumen: 4.8 per cent of total sample; 4.2 per cent theoretical requirement.

Test No. 101-VIII-S, Bd. 31-D. Crushed gravel treated by mixing method; bituminous surface, 1¾ inches thick

Grading of total sample		Grading of material finer than 10-mesh	
Size	Per cent passing	Size	Per cent passing
200-mesh.....	13.5	200-mesh.....	23.6
10-mesh.....	55.9	100-mesh.....	40.1
3-mesh.....	73.5	50-mesh.....	57.1
1½-inch.....	83.4	40-mesh.....	64.5
¾-inch.....	96.4	30-mesh.....	70.6
1-inch.....	100.0	20-mesh.....	82.5
		10-mesh.....	100.0

Bitumen: 4 per cent of total sample; 4.3 per cent of theoretical requirement.

Test No. 105-VIII-S, Bd. 31-E. Crushed gravel treated by mixing method; bituminous surface, 2¾ inches thick, is hard, tight, and smooth

Grading of total sample		Grading of material finer than 10-mesh	
Size	Per cent passing	Size	Per cent passing
200-mesh.....	9.1	200-mesh.....	20.5
10-mesh.....	49.5	100-mesh.....	37.5
3-mesh.....	66.0	50-mesh.....	55.9
1½-inch.....	76.7	40-mesh.....	63.8
¾-inch.....	93.4	30-mesh.....	70.0
1-inch.....	100.0	20-mesh.....	87.4
		10-mesh.....	100.0

Bitumen: 3.5 per cent of total sample; 3.5 per cent theoretical requirement.

FIG. 32.—TYPICAL OIL STAINS

mixing 4 gallons of oil per square yard with the earth. The analysis shows a very high percentage of fines and the proportion of oil to sand is 9.4 per cent, which agrees very closely with the 9.2 per cent determined as the theoretical requirement by means of the formula mentioned above. The stain is typical of that produced where the amount of bitumen is just sufficient to coat the particles and to secure the necessary binding.

Test 110 represents material from the oil-treated crushed gravel surface between Brawley and Imperial. The sand in this sample is somewhat coarser than in test 106 and the amount of bitumen required is, therefore, less. A satisfactory stain produced in this case, by oil in the amount of 8.4 per cent of the material passing the 10-mesh sieve, indicates that percentage to be the correct amount of oil.

Test 111 was taken from the oiled sand shoulders adjacent to the pavement on the Sand Hills section between Holtville and Yuma. The analysis shows a very small percentage of dust passing the 200-mesh sieve and a moderate percentage passing the 100-mesh. The amount of oil is about 3 per cent higher than in test 106 and the very heavy stain clearly shows an excess. The suggested remedy would be to add fine

material, either dust or sand, in sufficient amount to dry the mix until a normal stain is produced.

Test 113 is from a sample of the oiled sand roadway constructed east of Meyers Creek Bridge in Imperial County. The analysis shows the sand to be much coarser than the other samples; in fact, closely approaching a concrete sand. Fines are almost entirely lacking. A normal stain in this case was produced by $4\frac{1}{2}$ per cent of oil. The road constructed with this material is satisfactory, and an inspection of the road shows that the amount of oil used was entirely sufficient.

In these samples three normal stains were produced with varying amounts of oil and the one extremely heavy stain was due to excess of oil and lack of dust in sand.

Figures 30 and 32 show other typical stains and analyses of samples. All of these samples with the exception of 107 are from surfacing which is giving satisfactory service and which apparently contains about the correct amount of oil. Test 107 was made on a sample of crushed gravel surface which had been reoiled and which appeared to contain more oil than is desirable. The stain corroborates the observations made at the time of the inspection.

SELECTION OF THE METHOD AND OIL FOR TREATMENT

From the foregoing description and analysis of the methods of surface treatment and surface mixing—the two methods of treating crushed rock and gravel roads thus far developed—it is apparent that the selection of one or the other for the treatment of a particular road is a matter requiring the exercise of seasoned judgment.

Method of surface treatment.—The method of surface treatment is adapted to compacted material only. Usually a road is in best condition for treatment after being subjected to traffic for a time. However, new surfacing material, if thoroughly compacted with water and traffic during construction, is very satisfactory. Being somewhat porous, such new material absorbs oil readily.

An old road containing considerable coarse material, if it can be repaired well in advance, is well adapted to surface treatment because the large stones are well anchored.

Method of surface mixing.—This treatment is adapted to any crushed rock or gravel road which is in need of the addition of considerable new material to restore its thickness, or which, because of lack of binder or climatic condition, is not tightly bound.

The surface-mixing method is the only one that is adapted to the treatment of a loose surface; but in the use of this method, as in surface treatment, the base course should be thoroughly compacted. The presence of material larger than 1 inch presents mechanical difficulties.

Where traffic can not be kept off the road during the time required for absorption of oil applied by the surface treatment method, the surface-mixing method is especially attractive. If the oil be partially mixed with the metal immediately after application, traffic can pass through without difficulty. The loose surface encourages slow driving, whereas the firm surface always present in the other type of treatment tempts rapid driving and the consequent spattering of oil on vehicles.

Many existing projects have lost so much material that renewal is required before treatment. If enough remains for a base, it may be repaired and shaped, covered with approximately 2 inches of material passing a three-fourths or 1-inch circular opening, and treated by the mixing process immediately.

Either type.—A well compacted, fine crushed rock or gravel surface of adequate thickness may be treated successfully by either method. Choice depends upon relative service and life, about which there is no definite information. The mixing treatment will cost more than the surface treatment, and, therefore, must show a correspondingly longer life or lower maintenance to justify its use. The surface treatment, if not thoroughly successful, can later be converted into a mixed treatment, with a final cost not greatly exceeding that of an original mixed treatment.

Emergency treatment.—A problem frequently arises with the discovery that untreated material is being dissipated so rapidly that a few months' delay will bring complete failure. Usually a large proportion of the original material has already been lost. If restoration to full original thickness or substitution of a heavier type of surfacing can not be done immediately, it may be expedient to oil the metal remaining to prevent its complete destruction. Some failures may be expected but can not reasonably be charged to the treatment.

OIL FOR SURFACE TREATMENTS AND SURFACE MIXING

In the treatments of crushed rock and gravel roads described in this report the bituminous binder has usually consisted of the material known locally on the Pacific coast as "fuel oil," or the lightest grades of "road oil." Both are asphaltic base products.

The material sold as "fuel oil" has an asphaltic content up to about 70 per cent and a viscosity, Saybolt-Furol, up to 300 at 122° F. The term "road oil" has been applied to oil residuum having an asphaltic content of 70 to 98 per cent, being thus distinguished from the

heavier residuums which are usually termed "asphalts" of E or D grade.

Asphalt content in this connection and as used in this report is the percentage of residue having a penetration of 80 at 77° F. as "fuel oils" have usually been specified by specific gravity in degrees Baumé, or by viscosity rather than by asphalt content.

"Fuel oils" are considerably cheaper than "road oils" or "asphalt," and they are especially appropriate for most of the work that has been described because they penetrate readily and have the advantage of being much more easily reworked than the heavier materials.

"E" grade asphalt and the 90 to 95 per cent road oils have been very extensively used in the Western States in bituminous macadam work and to a more limited extent in surface-treatment work. Some of the cities and counties of California have secured very satisfactory results by building a well-bound coarse rock macadam the surface of which has been thoroughly broomed, then covered with road oil and screenings. Somewhat similar treatments have been widely used by various highway departments. Los Angeles, Orange, Monterey, San Joaquin, San Benito, Santa Cruz, San Bernardino, and other California counties have used large quantities of road oils in surface and penetration treatments with marked success. This report, however, is devoted to the use of "fuel oil" or the lightest grades of "road oil." "Fuel oils" are considerably cheaper than the "road oils" or asphalt, penetrate readily, and are more easily reworked.

Viscosity may vary widely independently of asphaltic content, which permits selection of various grades for special uses while specifying a single range of asphalt content, as 60 to 70 per cent. Maximum penetration can be secured by using oil of low viscosity. For mixed treatments medium viscosity appears desirable, because the oils with high viscosity are harder to handle and more difficult to mix with road metal. In surface-treatment work where chips are to be used in quantity high viscosity is desirable.

Probable rainfall is another consideration affecting the selection of oil. The lightest oil shows a tendency to wash out of the road during long-continued wet weather. In many instances good results have been secured by applying light oil as a first treatment, followed, after this treatment has penetrated, by an application of heavy oil and screenings. In all of these matters viscosity is an important characteristic.

All of the grades of "fuel oil" used in bituminous treatment are more uniformly distributed if heated to 150° F. or 200° F. The oil is usually heated with steam at the time it is unloaded from the car.

Following is a suggested specification which has been used by the California Highway Commission for the purchase of oil for 1927 operations. It provides "fuel oil" of three grades adaptable for use as outlined above. These specifications have been discussed with repre-

sentatives of the large producers of California oil and are believed to exclude any unnecessary requirements which would be reflected in higher costs.

SPECIFICATIONS FOR FUEL OIL FOR USE IN BITUMINOUS TREATMENTS

Oil furnished under these specifications shall be made from an asphaltic-base oil. It shall be either a naturally homogenous oil or a homogenous residue from oil. It shall not have been distilled at a temperature high enough to injure by burning or to produce flecks of carbonaceous matter. It shall meet the following requirements:

Flash point.—The flash point by the Pensky-Martens closed-tester method shall not be lower than 175° F.

Viscosity.—Oil shall be classified as to viscosity in three grades as follows:

Light fuel oil shall have specific viscosity (Engler) at 122° F. of not less than 10 nor more than 25.

Medium fuel oil shall have a specific viscosity (Engler) at 122° F. of not less than 25 nor more than 45.

Heavy fuel oil shall have a specific viscosity (Engler) at 122° F. of not less than 45 nor more than 80.

Water and sediment.—Water and sediment combined shall not exceed 2 per cent.

Asphalt content.—Fuel oil shall contain not less than 60 per cent nor more than 70 per cent of asphaltic residue having a penetration of 80 at 77° F.

Test methods.—Except as hereinafter specified, all tests of fuel oils shall be made in accordance with standards of the American Society for Testing Materials applicable thereto, with amendments and revisions thereof adopted by said society. The following special test requirements are noted for viscosity and asphalt determinations:

VISCOSITY

Specific viscosity (Engler) shall be determined on a 50-cubic centimeter sample in the manner described in United States Department of Agriculture Bulletin No. 1216, page 59. The determination of viscosity may also be made, at the option of the producer, in accordance with the A. S. T. M. viscosity test D 88-26, using the Saybolt-Furol viscosimeter at a temperature of 122° F., in which case the Saybolt-Furol time shall be not less than 35 nor more than 90 seconds for the light fuel oil; not less than 90 nor more than 160 seconds for medium fuel oil; and not less than 160 nor more than 280 seconds for heavy fuel oil.

ASPHALT CONTENT

Asphalt content shall be determined by open evaporation using the A. S. T. M. tentative method D 243-26 T, the open-cup vapor-bath method as used in the laboratory of the California Highway Commission, or the Brown evaporator.

TREATMENT OF EARTH ROADS WITH OIL

The earliest oil treatment in California consisted of the application of crude asphaltic oil, approximately a gallon per square yard, to the surface of a dirt road. The oil was allowed to penetrate at will or dirt was thrown on top. This was virtually a surface treatment and generally proved inadequate except for very light traffic or where the subgrade was unusually firm. The next development was the reworking of the thin mat, adding more oil and stirring to a greater depth. This was the crude beginning of the mixing treatment. The process was repeated in later years until some of the roads first treated had a surface 6 inches or more in thickness.

Paralleling the stage development, some of the counties have in the first operation deliberately built an oiled earth surface from 4 to 8 inches in thickness.

The success or failure of these oiled earth roads depends largely upon the character of the material. Sandy, gravelly, or even silty soils have produced serviceable roads in most instances where properly treated and maintained. Heavy clays or alkali soils have been similarly treated but the results have been almost universally unsatisfactory. These clays are inherently unstable, have high moisture-carrying capacity, and are constantly changing in volume. Uniformity of soil from point to point on the project is highly desirable, but experienced workmen will overcome considerable irregularity of soil by varying the quantity of oil and the details of treatment.

Under California conditions the cost of treating an earth road 18 feet wide to a depth of 6 inches will approximate \$3,000 or \$3,500 per mile. The cost of a treatment 1 to 2 inches in depth, where conditions make such a light treatment satisfactory, is \$200 to \$500 per mile. Where the full depth of 6 inches or more is attained by progressive steps, the \$3,000 cost will perhaps be reached in 8 or 10 years or more, this including the cost of maintenance during that period.

Heretofore the amount of oil for a given depth of treatment has been judged by the appearance of the product. The amount required depends, of course, upon the grading of material and probably follows the rules applicable to satisfactory sheet asphalt or asphaltic concrete, excepting that the lighter oil used in soil treatment covers the particles with a thinner film than the ordinary paving grades of asphalt. The amount of oil required is about one-fourth less than the amount of asphalt needed for the same grading. Samples show an oil content ranging from 4.5 to 10 per cent based upon the material passing a 10-mesh sieve. Reference is made to the discussion of the tests of surface-mixed material for more detailed information and suggestion as to the proportion of oil, and particularly to Figure 31.

This type of treatment is adapted to roads carrying less than 500 vehicles per day where soil conditions are suitable and particularly in arid or semiarid climates. The economy of omitting expensive road metal is obvious, and that possibility should always be considered. The oiled sand road in the Amargosa Desert in Nevada is typical. Here graveling was attempted and abandoned because of construction difficulties. The natural sand road was then oil treated and the surface obtained is as satisfactory as if the road had first been metaled.

Maintenance is a simple operation with blade and drag. Riding qualities are unexcelled and there is, of course, no dust. When reworking is required, a tractor, scarifier, and heavy blade grader must be available.

The examples and methods described hereafter are regarded as typical of oiled earth construction under somewhat varying conditions.

OILED SAND ROADS IN STANISLAUS COUNTY

Oiled earth roads have been successfully constructed in Stanislaus County, Calif., for about 18 years. The soil in this area is fine sand and sandy silt. Without treatment the roads are extremely dusty and loose during the summer season.

The Turlock-Snelling road, extending east from Turlock for a distance of about 3 miles, is a typical example of this construction. The surfacing has a width of about 20 feet and a depth of 6 to 7 inches. The material is plastic under a hammer and more or less rubbery. The contour is smooth, and some gravel is visible which has probably been placed during maintenance. In a number of places fine surface cracks were noted during an inspection in July, 1926, but examination showed these to be only one-eighth to one-quarter inch in depth through a thin hard crust which covered the softer plastic material beneath. The asphaltic oil in a sample cut from the road was alive and tacky. An analysis of this sample gave the following results:

	Per cent
Passing ½-inch screen.....	98.7
Passing 10-mesh sieve.....	98.5

The material passing the 10-mesh sieve was considerably finer than an ordinary sheet asphaltic mixture and the amount of silt slightly more than the dust filler in such a sheet asphaltic mix. It had the following grading:

	Per cent
Passing 10-mesh sieve.....	100.0
Passing 20-mesh sieve.....	99.9
Passing 30-mesh sieve.....	97.4
Passing 40-mesh sieve.....	93.1
Passing 50-mesh sieve.....	84.1
Passing 100-mesh sieve.....	47.1
Passing 200-mesh sieve.....	23.8
Bituminous material.....	7.9

The following statement regarding the construction of this highway is abstracted from a report by A. J. Leedom, county superintendent, who has been in charge of construction. The natural soil was graded to a flat section about 26 feet in width with very little crown, was loosened on top, and from 5 to 6 gallons of oil per square yard were spread in two equal applications. The first application covered a width of about 20 feet, and immediately the entire surface was plowed, starting at the edges and throwing furrows out, finally leaving a dead furrow in the center. This gave a roadway width of about 22 feet. The plowing was generally to a depth of about 6 inches. The road was then harrowed, winding back and forth in order to break up the furrows and thoroughly mix the oil and earth. The second application of oil was spread, with more at the edges than in the center to prevent excessive concentration of oil at the center after the next operation. The entire surface was then replowed,

throwing furrows to the center and bringing the road to a uniform section, harrowed as before, and finished with a steel drag, 7 or 8 feet long and weighing about 600 pounds.

The bituminous material was California asphaltic oil residuum containing about 65 per cent of 80-penetration asphalt, and was applied from a gravity spreader at a temperature somewhat less than 200° F.

The Turlock-Snellings road is about 3 miles in length and has been in service for 17 years. The traffic was undoubtedly light during the early years, but has now grown to approximately 500 vehicles per day, and the maintenance cost is reported to be less than \$100 per mile per year. The original cost of the work by county labor was about \$3,500 per mile.



FIG. 33.—TYPICAL SECTION OF THE TURLOCK-SNELLINGS ROAD. THE SANDY SOIL IS PROBABLY RESPONSIBLE FOR THE SUCCESS OF THE ROAD IN SPITE OF LACK OF DRAINAGE

Roughometer readings on the surface thus produced show a smoothness about equal to average asphaltic macadam. It is entirely dustless and to the casual observer might easily pass for a higher type of bituminous construction. The low construction cost and extremely low maintenance combine to give an annual service charge among the lowest for improved roadway investigated in this study. Several other sections of roadway in this locality constructed by the same methods and varying in age from 1 to 12 years, where sufficiently wide to keep traffic from breaking the edges, were found to be uniformly satisfactory. Some of the older roads, oiled to a width of 12 feet, had ragged edges and somewhat irregular surfaces. A photograph of such a road is shown in Figure 33.

OILED EARTH ROADS IN RIVERSIDE COUNTY

Riverside County, Calif., constructed several experimental sections of oiled earth roads during the years 1924, 1925, and 1926, similar to the earlier work in Stanislaus County. Twelve and one-half miles was in the irrigated section of the Coachella Valley, where the soil is fine sandy silt, the climate extremely dry, and the roads almost impassable during the crop-hauling season.

The earlier projects were treated with 4 gallons of oil having a 60 per cent asphaltic content (80 penetration). The viscosity is not known, but the oil was fluid at temperatures of 100° to 150° F. The oil was spread in applications of 1 gallon per square yard, in widths of 6 and 8 feet, the finished width of roadway being 14 feet. Distribution was accomplished by a truck equipped with pressure sprays, towed through the loose soil by a

5-ton track-type tractor. Two sets of four-gang disks followed the spreader and mixed the oil with the earth. This operation was repeated through four applications of 1 gallon each and the oil was mixed with the earth to a depth of about 5 inches. In the later projects the oil was increased to 5 gallons and cultivated to a depth of 6 inches. After cultivation, the surface was smoothed with a grader and rolled with a 10-ton power roller. In some instances the roads were opened to traffic before rolling. William S. Conner, the engineer in charge of this work, expresses the opinion that extensive rolling is unnecessary and that consolidation can best be obtained by permitting traffic to use the road, provided it is kept smooth by drags.

The surface resulting from this treatment was practically identical with that in Stanislaus County where soil conditions were similar. On the Ripley-Blythe Road, where the soil contained a considerable percentage of clay, 4 gallons of oil did not produce uniformly satisfactory results, and the surface remained more or less unstable.

The Riverside County work is too recent to be interesting from a service standpoint, but is valuable because of the excellent cost data furnished by Mr. Conner. Table 12 shows the average cost to be practically \$3,000 per mile exclusive of clearing and grading.

During an inspection in February, 1927, it was found that several sections had been resurfaced with about 1 inch of crushed stone, treated with 90 to 95 per cent road oil, and covered with chips. Sections not metaled are carrying traffic satisfactorily although they do not have the appearance of being maintained.

OILED SAND ROADS IN SAN BERNARDINO COUNTY

J. S. Bright, construction engineer, Bureau of Public Roads, gives data regarding the oiling of desert roads under his direction as county highway engineer of San Bernardino County in 1916. The natural road material in this case was sand. Intermittent sections of the Barstow-Needles road, aggregating about 40 miles, were treated to a width of 8 feet with 2½ gallons per square yard of oil having an asphalt content of 60 per cent. The road received very little attention until taken over by the State highway commission in 1924. Traffic, which at first amounted to 100 or more vehicles daily, has since increased to about 400 and the surface has been practically destroyed in many places.

One section, extending from Amboy easterly for about 15 miles, received 4 to 4½ gallons of oil per square yard on a width of 10 feet. The resulting surface was 5 to 6 inches in depth and is still in fair condition, although it received practically no maintenance for the first eight years. The natural material is a gravelly sand considerably coarser than that treated in Riverside and Stanislaus Counties. Samples were removed for test in February, 1927, and the oil was found to be alive and sticky. It was found that 30.9 per cent of the total was retained on the 10-mesh sieve and consisted of coarse sand, mostly less than one-fourth inch in diameter. The portion passing the 10-mesh sieve was graded as follows:

	Per cent
Passing 10-mesh sieve.....	100.0
Passing 20-mesh sieve.....	71.6
Passing 30-mesh sieve.....	42.5
Passing 40-mesh sieve.....	37.0
Passing 50-mesh sieve.....	31.6
Passing 100-mesh sieve.....	22.4
Passing 200-mesh sieve.....	11.0
Bituminous material.....	8.1

TABLE 12.—Costs per mile of 14-foot oiled earth road in Riverside County, Calif.

Item	1924-1925 treatments			1925-1926 treatments					
	Ripley-Blythe Road	Avenue 62	Average	Morgan-Ickes Road	Avenue 58 and Monroe Street	Avenue 57 and Pierce Street	Jackson Street	Avenue 66	Average
Length.....miles.....	8.13	2.00		1.51	2.00	1.00	2.00	4.00	
Oil.....gallons per square yard.....	3.83	3.95	3.85	5.19	4.92	4.92	4.92	4.92	4.96
Average haul of oil.....miles.....	2.4	7.0	3.3	8.0	7.0	2.25	4.5	10.0	7.36
Labor:									
Clearing and grading.....	\$234.10	\$285.05	\$244.16	\$671.44	\$351.85	\$415.59	\$353.53	\$421.43	\$430.64
Watering subgrade.....	141.88	226.08	158.51	72.85	131.12		100.27	100.22	92.64
Hauling and spreading oil.....	359.17	501.30	387.22	602.81	573.38	368.60	434.55	421.50	473.85
Cultivating.....	122.69	132.06	124.54	82.99	82.13	92.50	142.41	73.15	91.29
Rolling.....	47.00	64.25	50.41	88.49	61.65	80.88	43.40	25.46	50.89
Dragging.....	36.82		29.55	2.65	44.15	7.86	2.58	14.28	25.35
Maintaining detour and miscellaneous.....	21.05	32.29	23.27	244.51	56.11	35.47	19.38	72.02	70.38
Supervision.....	55.72	94.45	63.31	242.82	79.86	69.55	48.19	45.50	83.19
Total (excluding grading and clearing).....	784.33	1,050.43	836.87	1,337.12	1,028.40	654.76	790.78	752.03	886.80
Materials:									
Oil.....	1,190.18	1,255.31	1,203.04	1,526.99	1,443.93	1,443.93	1,443.93	1,443.95	1,455.87
Freight.....	996.24	523.43	902.89	714.02	675.18	675.18	675.18	706.44	692.66
Miscellaneous.....	15.78	13.44	15.32	49.67	27.23	65.00	27.50		23.74
Total.....	2,202.20	1,792.18	2,121.25	2,290.68	2,146.34	2,184.11	2,146.61	2,150.39	2,172.26
Grand total cost.....	3,220.63	3,127.66	3,202.28	4,299.24	3,526.59	3,254.46	3,290.92	3,323.85	3,489.70

The condition of the Amboy section after 11 years of rather heavy service almost without attention indicates that similar construction of adequate width, properly maintained, would carry traffic very satisfactorily over the desert sections. The cost of such treatment would probably be about the same as reported in Riverside County.

OILED-SAND SURFACE ON NEVADA STATE HIGHWAY

A 10-mile section of the Las Vegas-Beatty road in the Amargosa Desert in Nevada is a recent example of oiled-sand construction. This section of highway, designated as Nevada Federal Aid Project 46, belongs to the same general type as the Amboy section in San Bernardino County, Calif. The difficulties encountered were extreme and unusual. The shifting sand was almost impassable for motor traffic prior to the improvement, and during construction the trucks could not travel through the sand without the assistance of tractors.

Late in 1922 bids were requested for the construction of gravel surfacing. Only one was received and that, being 84 per cent higher than the engineers' estimate, was rejected. In January, 1923, the State highway department undertook the work by day labor and opened a gravel pit near the middle of the 10-mile project. Because of the isolation of the job, the extremely hot weather, and transportation difficulties the work was discontinued after five months with but 2 miles partially constructed. The cost of gravel being prohibitive, early in 1924 the department undertook to mix asphaltic oil with the natural soil to a depth of approximately 6 inches. The oil used for mixing purposes had an asphalt content of 61 to 64.5 per cent of 80-penetration. The roadway was to be trenched to a depth of approximately 6 inches, and the material removed from the trench was piled along the shoulders to be used for covering the oil. Shortly after beginning work the depth of trenching was modified to approximately 3 inches, as the casting of sand upon the oil was found unnecessary and uneconomical except in small quantities.

After the first application of oil upon the prepared subgrade the roadway was cultivated with a double-action disk harrow. A second application of oil followed and was partially covered with sand from the

shoulders to prevent the oil from collecting in large pools. Disking was resumed, followed by a large road grader which moved the oiled material into a windrow first on one side of the road and then on the other, and finally leveled it for the third application. The grader was the most effective implement used for mixing. This procedure was repeated until the required amount of oil had been applied and mixed. On most of the project four applications were spread. Table 13 shows data regarding amounts of oil used and number of treatments.

When the surface became uniform in appearance, it was shaped with the grader to a crown approximating 2 inches and compacted with a sheeps-foot roller. For the final smoothing, a drag 10 feet wide by 20 feet long was used in conjunction with an ordinary roller 3 feet in diameter having a weight of approximately 175 pounds per inch of width.

After the mixing was completed and the surface compacted, part of the project was covered with road oil containing 85 per cent of asphalt (80 penetration). This treatment, as shown in Table 13, was varied according to the condition of the surface, one section receiving the maximum of 1.30 gallons per square yard.

The plans called for a roadway 10 feet in width with 18 turnouts. Actually the width averages 12 feet because of a general squeezing outward during construction and under traffic before the mixture hardened.

The cost, shown in Table 14, averaged practically \$5,000 per mile, or \$0.82 per square yard. This is above normal because of the remoteness of the work. Oil was shipped from Los Angeles in tank cars, and was hauled an average of 19 miles from the railroad in distributor trucks having a capacity of 600 to 800 gallons. The work was carried on during extremely hot weather, so hot, in fact, that the light oil required no artificial heating.

The 2-mile section which had previously been graveled received the same treatment as the desert sand, except that the amount of oil was cut approximately in half and the thickness of the oil mixture proportionately reduced.

Two years after completion the surface was examined for width and thickness, and six samples were analyzed. The results are shown in Table 15. The oil content ranged from 6.40 to 8.47 per cent, averaging 7.40 per cent. The sand grading resembles that of sheet asphalt.

TABLE 13. *Distribution of oil on Nevada Federal-aid project No. 46—an oiled-sand road*

Station to station	Length	Area of surface ¹	Quantity of oil applied		Number of applications		Total quantity applied		Kind of material in roadbed
			Total	Per square yard	61-64.5 per cent asphalt	85 per cent asphalt	61-64.5 per cent asphalt	85 per cent asphalt	
	<i>Feet</i>	<i>Sq. yds.</i>	<i>Gals.</i>	<i>Gals.</i>			<i>Gals. per sq. yd.</i>	<i>Gals. per sq. yd.</i>	
765+00-794+50	2,950	3,278	15,805	4.82	4	1	4.15	0.67	Sand.
794+50-812+00	1,750	1,944	11,098	5.71	4	2	4.41	1.30	Do.
800+00-878+00	1,800	2,000	9,603	4.80	4	1	4.12	.68	Do.
878+00-916+00	3,800	4,222	18,291	4.33	4		4.33		Do.
916+00-930+00	1,400	1,556	7,223	4.64	4	1	4.11	.53	Do.
930+00-935+35	535	594	3,411	5.74	5		5.74		Do.
935+35-941+45	610	678	3,000	4.42	4		4.42		Do.
941+45-973+00	3,155	3,506	11,800	3.37	3		3.37		Do.
973+00-1022+10	4,910	5,455	22,412	4.11	3	1	3.50	.61	Do.
1022+10-1040+00	1,790	1,989	9,830	4.94	4	1	4.25	.69	Do.
1040+00-1050+00	1,000	1,111	4,396	3.96	3	1	3.27	.69	Do.
1050+00-1081+90	3,190	3,544	15,540	4.38	3	1	3.65	.73	Gravel.
1081+90-1085+88	398	442	1,536	3.47	2	1	2.59	.88	Do.
1085+88-1087+80	192	213	479	2.25	1	1	1.47	.78	Do.
1087+80-1122+00	3,420	3,800	15,589	4.10	3	1	3.31	.79	Do.
1122+00-1123+00	100	111	85	.77	1			.77	Do.
1123+00-1127+00	400	444	1,741	3.92	3	1	3.15	.77	Do.
1127+00-1210+00	8,300	9,222	13,701	1.48	1	1	.64	.84	Do.
1210+00-1227+00	1,700	1,889	2,807	1.49	1	1	.48	1.01	Sand.
1227+00-1235+35	835	928	1,863	2.01	2	1	1.18	.83	Do.
1235+35-1240+00	465	516	865	1.68	1	1	.87	.81	Do.
1240+00-1284+20	4,420	4,911	29,593	6.02	5	1	5.41	.61	Do.
1284+20-1297+46	1,326	1,473	6,589	4.47	4	1	4.01	.46	Do.
1297+46-1312+00	1,454	1,616	9,454	5.85	5	1	5.39	.46	Do.
1312+00-1343+24	3,124	3,417	16,914	4.96	5		4.96		Do.
Total	53,024	58,913	233,625						
Average				3.97			3.39	.58	

¹ Surface assumed to be 10 feet wide. Actual width greater, due to turnouts and spreading of soft material under traffic.

The roadway produced meets traffic requirements. It is smooth, and with moderate maintenance, will probably be durable. It is not as dustless as anticipated, because a film of sand has been kept on the surface to absorb surplus oil. Evidently the heavy oil last applied has furnished unnecessary richness at the surface. Evidence is lacking that it added to the quality of the work.

EMERGENCY OILED SAND ROAD IN IMPERIAL COUNTY

In December, 1926, a heavy flood destroyed nearly 2 miles of concrete pavement on the San Diego-El Centro road about 30 miles west of El Centro, Calif. The damaged road was paralleled by a broad expanse of sand in the Myers Canyon Wash, a strip of which was promptly oiled to serve as a detour pending reconstruction. Nominally 5 gallons per square yard of 60 to 70 per cent "fuel oil" was spread, but tests indicate that the actual rate of application was somewhat less than 4 gallons per square yard; the excess having been used to attain extra width. The oil was mixed with the sand by cultivation and grading to a depth of about 7½ inches. By the end of January,

TABLE 14.—*Summary of cost of Nevada Federal-aid project No. 46—an oiled sand road.*

Item	Cost per mile		
	Labor	Equipment and supplies	Total
Preparing subgrade	\$25.19	\$65.00	\$90.19
Trenching for surface	93.85	162.00	255.85
Applying oil	829.92	1,406.00	2,235.92
Oil		1,788.39	1,788.39
Applying sand and cultivating	257.16	180.70	437.86
Dragging and final shaping	90.21	86.70	176.91
Signs		10.46	10.46
Total	1,296.33	3,699.25	4,995.58
Cost per square yard (6,100 square yards per mile)			.82

TABLE 15.—*Analysis of samples from Nevada Federal-aid project No. 46—an oiled-sand road*

[Samples taken in June, 1926, two years after construction]

Station	Dimensions of section		Quantity of oil applied (gallons per square yard)	Fraction of aggregate passing various sieves (per cent)								Oil content (per cent)	
	Width	Depth		10-mesh	14-mesh	20-mesh	30-mesh	40-mesh	50-mesh	80-mesh	100-mesh		200-mesh
	<i>Ft.</i>	<i>Ins.</i>											
770	12.0	5.75	4.82	95.8	95.3	94.8	90.9	82.5	69.7	45.7	27.3	8.9	8.47
800	13.0	5.05	5.71	98.3	97.9	97.5	94.4	84.3	67.9	43.7	25.4	7.8	7.27
900	11.5	7.05	4.33	90.9	89.7	88.8	85.1	75.8	63.9	43.6	26.9	8.5	6.40
950	11.0	3.65	3.39	92.5	90.7	88.8	84.5	75.2	62.2	44.6	29.8	11.2	8.20
1045	12.5	4.60	3.40	93.0	91.5	89.9	84.8	76.3	66.7	49.6	34.9	15.5	7.32
1250	12.0	6.45	6.02	87.3	86.2	85.1	83.4	75.4	63.8	42.2	25.0	6.7	6.73
Mean	12.0	5.42	4.61	91.3				78.2		44.9	28.2	9.8	17.40

¹ Equals 8.1 per cent of aggregate passing 10-mesh sieve.

1927, the surface had attained a fair degree of consolidation. On February 22 the center was supporting heavily loaded trucks and ordinary automobiles were running over the detour at 40 miles an hour. The edges, subjected to comparatively little traffic, were still soft.

A sample was taken on February 13 for test. All material passed the three-quarter-inch screen, 97.9 per cent passed the No. 3 screen, and 79.7 per cent passed the No. 10 sieve. The 10-mesh material had the following grading:

	Per cent
Passing 10-mesh sieve	100.0
Passing 20-mesh sieve	61.0
Passing 30-mesh sieve	36.5
Passing 40-mesh sieve	26.6
Passing 50-mesh sieve	18.2
Passing 100-mesh sieve	8.6
Passing 200-mesh sieve	4.9

The oil content was 3.6 per cent of the total, or 4.5 per cent of the 10-mesh material.

The sand in this temporary road is coarser than any other treated soil described. The amount of silt and clay is negligible. The oil content is low, but the appearance of the road shows that ample was used.

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FISCAL YEAR 1928

PROJECTS COMPLETED
JULY 1 1927

ITS COMPLETED SINCE
JUNE 30, 1927

PROJECTS UNDER CONSTRUCTION

PROJECTS APPROVED FOR
CONSTRUCTION

BALANCE OF
FEDERAL
AID FUND
AVAILABLE
FOR NEW
STATES

[illegible]

* Includes projects reported completed (final vouchers not yet paid) totaling:

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UNITED STATES DEPARTMENT OF AGRICULTURE
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THE DELAWARE RIVER BRIDGE AT PHILADELPHIA

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A JOURNAL OF HIGHWAY RESEARCH

U. S. DEPARTMENT OF AGRICULTURE

BUREAU OF PUBLIC ROADS

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TESTS OF THE DELAWARE RIVER BRIDGE FLOOR SLABS

By the Division of Tests, United States Bureau of Public Roads.

Reported by GEORGE W. DAVIS, Assistant Engineer of Tests

CONCRETE floor slabs, similar in all respects to those used in the construction of the Delaware River Bridge at Philadelphia and which are of special and unusual design have been subjected to a complete set of tests by the Bureau of Public Roads in cooperation with the Delaware River Bridge Joint Commission. The slabs were designed to reduce to a minimum the weight of the floor system of the Delaware River Bridge which at the time of its opening was the longest suspension span in the world. It has a main span of 1,750 feet and two side spans of about 720 feet each or a total length, including approaches, of 1.8 miles. The main and side spans are supported by two main cables, each 30 inches in diameter, from which are suspended the stiffening trusses which, in turn, support the floor system.

In a bridge of such magnitude it is obvious that the reduction of its dead weight to a minimum is a major feature of design. The structure is primarily a highway bridge and the dead weight is largely that of the highway deck. The design prepared for this portion of the structure, a reinforced concrete slab with a bituminous wearing surface, differed from previous practice to such an extent that tests were undertaken to determine the strength and behavior. These tests have confirmed theoretical conclusions as to the adequacy of the design and yielded data of value in designing similar floor systems. The tests were made at the Arlington Experimental Station of the bureau.

BRIDGE FLOOR OF UNUSUAL DESIGN

The main cables of the bridge are 89 feet apart. From them is suspended a floor system 125 feet 6 inches in width. In the center of this is a 57-foot roadway. On each side of the roadway is space for two electric car lines, one inside and one outside of each of the stiffening trusses, which hang directly under the main cables. Overhead footwalks, 10 feet in width, are carried on cantilever brackets on the outside of the trusses.

The floor system consists of steel floor beams riveted to each panel point of the stiffening trusses, these floor beams being 20 feet 6 inches center to center in the main span and 20 feet 8 inches center to center in the side spans. Longitudinal 18-inch I-beam stringers between the floor beams and spaced 3 feet 10 inches on centers support the concrete floor slab. Expansion joints are provided at every second floor beam.

The floor slab rests directly on the stringers. It is designed for a concentrated wheel load of 15,000 pounds with an allowance of 50 per cent for impact. On the suspended spans it is 6 inches in thickness and is covered with a 2½-inch asphalt wearing surface. The transverse or main slab reinforcement consists of Rivet-grip fabricated trusses spaced 6 inches on centers. The chords of these trusses are small rolled channels spaced 4½ inches back to back and the web bars are flats rolled with projecting lugs on the sides. In the process of manufacture the web bars were first bent to the required shape and then assembled with the chord channels and passed through a press which caused the flanges of the channels to grip the lugs of the web bars.

The longitudinal slab reinforcement consists of ½-inch round deformed bars spaced 6 inches on centers in the bottom of the slab and 12 inches on centers in the top. The reinforcement has a ¾-inch cover top and bottom. The concrete used was a 1:1½:3 mix.¹

The tests at Arlington were made upon two floor sections of similar design and approximately the same span length but of about one-third the width. The test slabs were supported by floor stringers resting upon concrete abutments instead of steel floor beams as in the bridge. Details of the test slabs are shown in Figure 1. All materials used in these slabs were identical with those used on the bridge and from the same sources.

The tests as run may be divided into two groups. The tests on the first slab were primarily proofing tests under both static loads and under impact to prove by actual physical tests that the slabs were adequate to withstand any probable loads to which they might be subjected.

The tests on the second slab were made to show the effect of the consistency of the concrete on the strength of the slab and to provide more exhaustive data as an aid to the design of slabs of this type. In this report each slab will be treated separately.

SCOPE OF TESTS ON FIRST SLAB

Materials used.—The aggregates used were Delaware River sand and gravel from above Bordentown, N. J.

The steel used consisted of Rivet-grip 4½-inch reinforcing trusses and ½-inch round deformed bars. (Fig. 2.) Test samples taken from the upper and lower chords of the reinforcing trusses showed a modulus of elasticity of 30,000,000 pounds per square inch.

The cement used showed a tensile strength of 315 pounds per square inch at the end of 7 days, and 355 pounds per square inch at the end of 28 days. Test cylinders from the concrete for the first slab showed a modulus of elasticity of 4,137,000 pounds per square inch and a compressive strength of 5,465 pounds per square inch at the end of 28 days, while at the end of 235 days a compressive strength of 5,975 pounds per square inch was developed.

Construction.—The slab was cast in a building which has a reinforced concrete floor 12 inches in thickness. This floor is supported on reinforced concrete beams 12 inches square on 8-foot centers both ways, carried on columns at beam intersections. With the slab in place no measurable deflection of the floor could be found by measurement with an Ames dial reading to 0.0001 inch, under a centrally applied static load of 30,000 pounds. The steel I beams under the test slab rested on concrete abutments 1 foot thick and 3 feet high built directly on the floor.

The mix used was 1:1½:3. The average slump was 2.9 inches using a 4 by 8 by 12 inch cone, and the average flow was 115.

¹ A more complete description of the floor system is contained in an article entitled "Reinforced Concrete Bridge Roadway on the Delaware River Bridge," published in *Concrete*, vol. 30, No. 2, February, 1927.

The concrete was mixed in a power-driven mixer for not less than two minutes, wheeled to the forms in wheelbarrows, dumped to a platform, and shoveled in place in the forms. The mixture was flowed into place, and air forced out by vibrating forms and reinforcement with an electric hammer provided with a flat-nosed bit and delivering 3,600 blows per minute. The slabs were covered with wet burlap immediately after the placing was completed. The forms which were wet before the concrete was placed were removed at the end of 24 hours. The burlap and the bottom of the slab were kept wet for seven days. The slab was 28 days old before any loads were applied to it.

During the tests on the first slab the following measurements were made:

- Deflection of the slab and the I beams under static loads.
- Deformations in the slab and in the I beams under static loads.
- Deformations in the reinforcing trusses under static loads.
- Spread of the two center I beams under static loads.
- Deflection of the slab under impact.
- Deformations in the slab and in the I beams under impact.

TEST INSTRUMENTS DESCRIBED

Static loads.—All static loads were applied at the center of the slab with a 150-ton hydraulic jack, which rested on a cast-iron block, which in turn rested on a rubber cushion consisting of three segments of a 30 by 4 inch solid rubber tire.

These tire segments were set in the direction of traffic (parallel to I beams). The jack reacted through an iron strut against two overhead I beams, 18 inches in depth, which were anchored to the foundations of the building by suitable steel work. The magnitude of the load was determined through the measured deflec-

tion of a calibrated beam. This deflection was measured with a 0.0001-inch Ames dial, the stiffness of the beam being such that one division on the Ames dial indicated a load of 185 pounds. Freedom of movement of the rider bar (fig. 3) on the calibrated beam and proper contact of the stem of the Ames dial was insured by vibration of the rider bar with an electric buzzer as the load was applied. Details of the static loading device are shown in Figures 3 and 4.

Deflections.—Deflections of the slab were determined by measuring the distance between gauge points on the bottom of the slab and I beams and corresponding points fixed on the concrete floor beneath with a wooden staff provided with a metal shoe at the lower end and an Ames dial at the upper end. The gauge points on the slab were small brass blocks, in which conical depressions had been drilled. These blocks were cast in place in the slab during construction.

The gauge points in the I beams consisted of conical depressions drilled in the I beams. The fixed points on the floor were similar conical depressions drilled in strips of 1 by $\frac{1}{4}$ inch steel bolted and grouted to the floor. A pair of steel blocks with like conical depressions were set into the concrete abutment at a fixed distance apart to serve as a standard to permit a check of the staff during the progress of the measurements. The layout of deflection points is shown in Figure 5.

Deformation and stress.—The deformations in the concrete and I beams were measured with 6-inch graphic strain gauges. This small device² has a system of levers so arranged as to magnify any movement of the gauge points at its ends about 75 times and

² More detailed information regarding this gauge and its operation may be found in the Engineering News-Record of Mar. 29, 1923, p. 575.

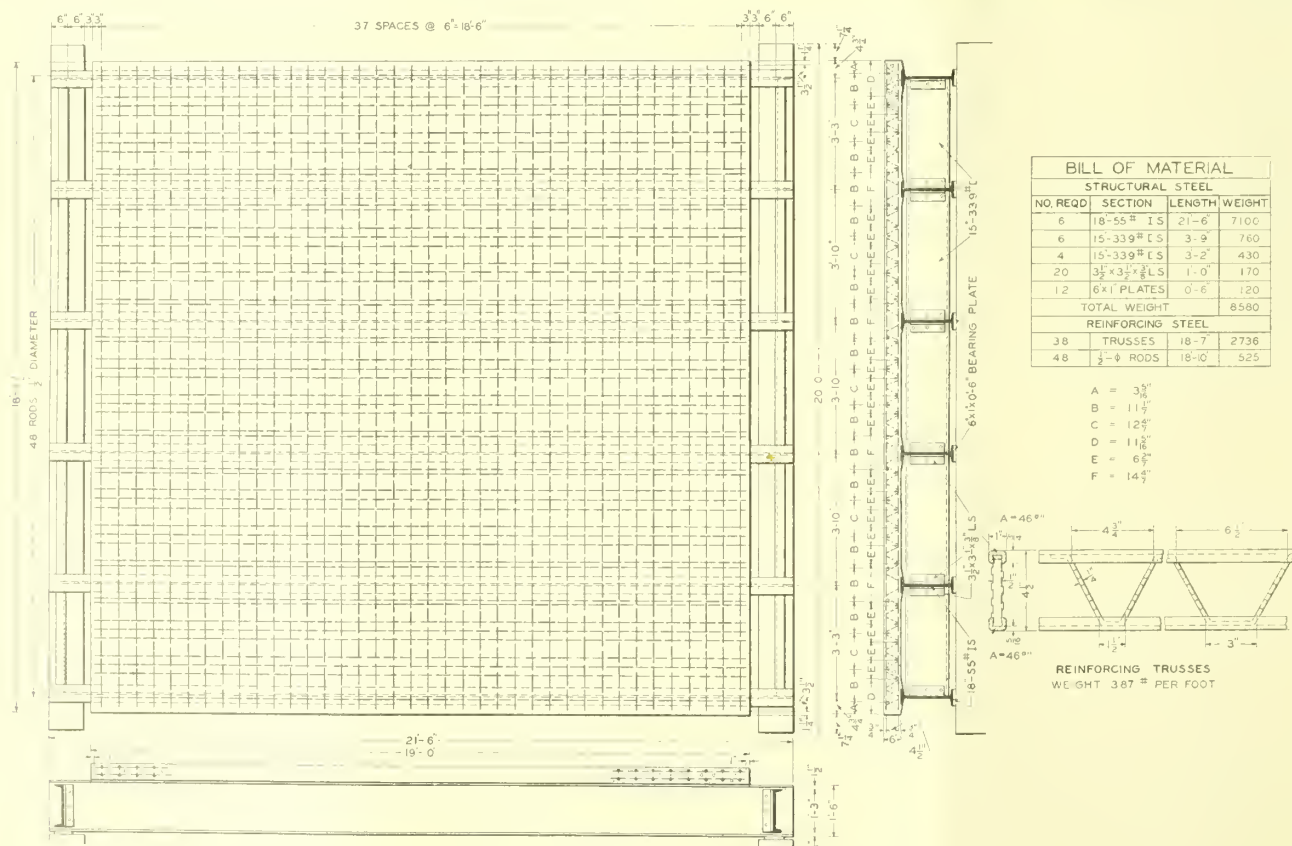


FIG. 1.—DESIGN OF TEST SLAB AND SUPPORTS

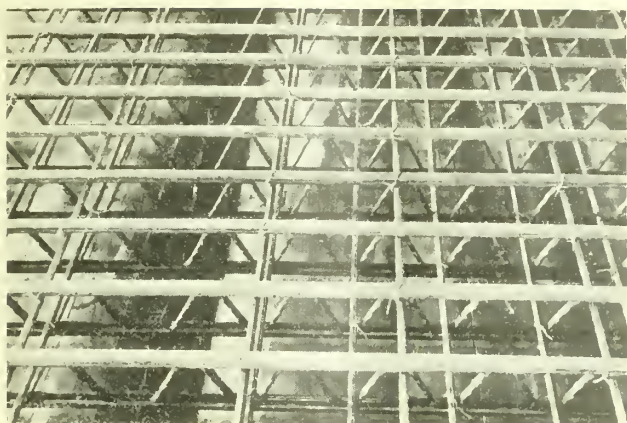


FIG. 2.—REINFORCING IN PLACE FOR TEST SLAB

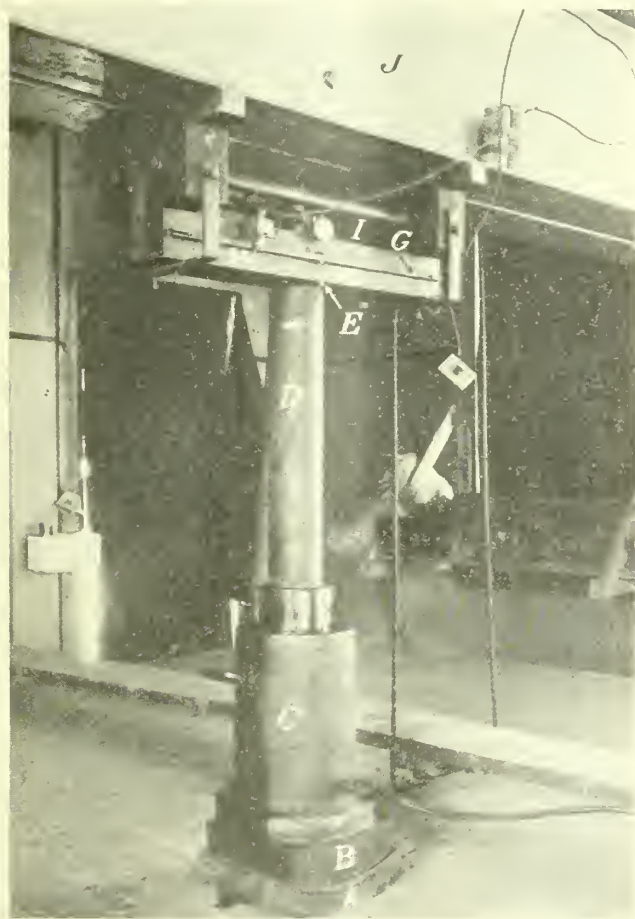


FIG. 3.—STATIC LOADING DEVICE—

A= rubber cushion. F= calibrated beam.
 B= cast-iron block. G= rider bar.
 C= hydraulic jack. H= buzzer.
 D= strut. I= Ames dial.
 E= knife-edge. J= overhead I beams.

records the movement on a smoked glass. It can be used to measure both contraction and expansion and the smoked glass can be adjusted to record several successive movements of the gauge points. The graphic record on the glass can be measured with suitable microscope equipment or may be projected on a screen with rectangular coordinates and the actual gauge point movement determined from measurement and the known constants of the instrument and projection apparatus.



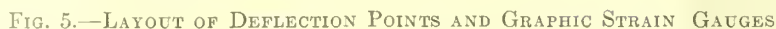
FIG. 4.—APPARATUS USED IN TESTING SLABS. THE DEVICE USED TO DETERMINE I-BEAM REACTIONS CAN BE SEEN AGAINST THE WALL

Gauges were placed on both bottom and top of slab and also on the webs of the I beams. The gauges on the slab were set between brass plugs set in the concrete. Those on the I beams were located 7 inches above and 7 inches below the axis of the beams and were set between brass plugs attached to the webs of the I beams by small bolts passing through the webs. Two 4-inch strain gauges were set at the center points on the bottom of the two center reinforcing trusses, between brass bearing plugs bolted to the lower chords of the trusses, slots being cut through the concrete to the steel. The layout of graphic strain gauges is shown in Figure 6 and some of them can be seen in Figure 4.

Spread of center I beams.—The spread, or the amount that the two center I-beam stringers were forced apart at their midspan by static loads, was measured by two wooden rods, each with an Ames dial mounted at one end and placed horizontally between the two I beams. Conical depressions were drilled in the webs of these I beams at the center of their span 3 inches below the top and 3 inches above the bottom of the beam, or on 12-inch centers. A wooden staff with a metal shoe on one end and an Ames dial on the other was placed between the two upper depressions and another similar staff between the two lower depressions. Both rods were supported by a framework attached to the I beam in such a way that they might move with the beams as they deflected.

Impact.—Impact was applied to the center point of the slab with the impact machine which has been used on previous research by the Bureau of Public Roads.³ This machine consists of a framework of structural steel on which is mounted a truck wheel fastened to a truck spring. Jackscrews at the corners of the frame and an electric motor suitably geared to two cams allow this wheel to be raised and suddenly dropped any desired distance. The sprung weight may be varied by changing the spring tension with two adjusting screws, while the unsprung weight may be varied by increasing or decreasing the weight of the free falling

³ See Public Roads, vol. 5, No. 2, April, 1924.



Deflection under impact.—In the measurement of the deflection of the slab under impact use was made of the same deflection points, both on the slab and the floor, that were used under static loading. Wooden rods with a metal shoe at the lower end and an Ames dial at the upper end were set up at selected deflection points, immediately under and surrounding the point of impact. Around the stem of each of these dials a

⁴ See *Public Road*, vol. 5, 10, December, 1924.

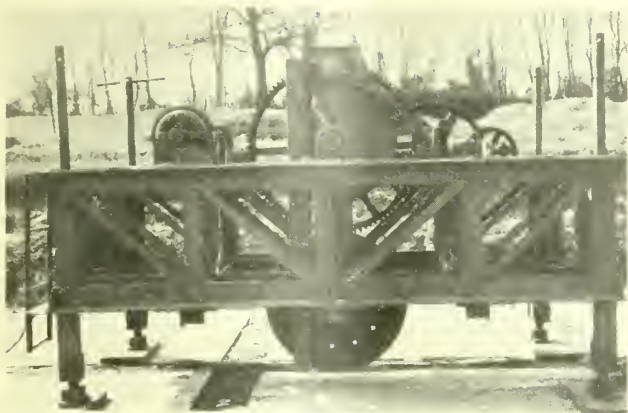


FIG. 7.—THE IMPACT MACHINE USED FOR IMPACT TESTS ON SLAB

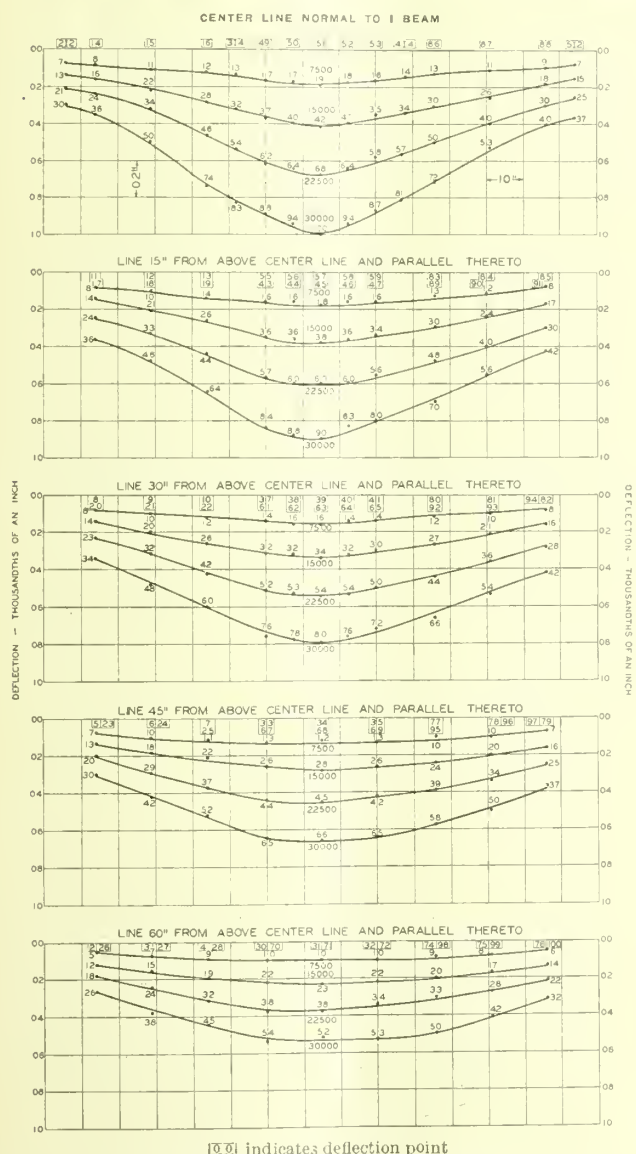


FIG. 8.—DEFLECTION OF FIRST SLAB UNDER STATIC LOADING BEFORE IMPACT

brass collar, fitted with a set screw, was placed. This set screw was tightened sufficiently to overcome, or choke, the action of the spring within the dial, but not to such an extent as to prevent movement of the plunger in the dial under a blow. The rods were set in place and all dials set at zero with the wheel of the impact

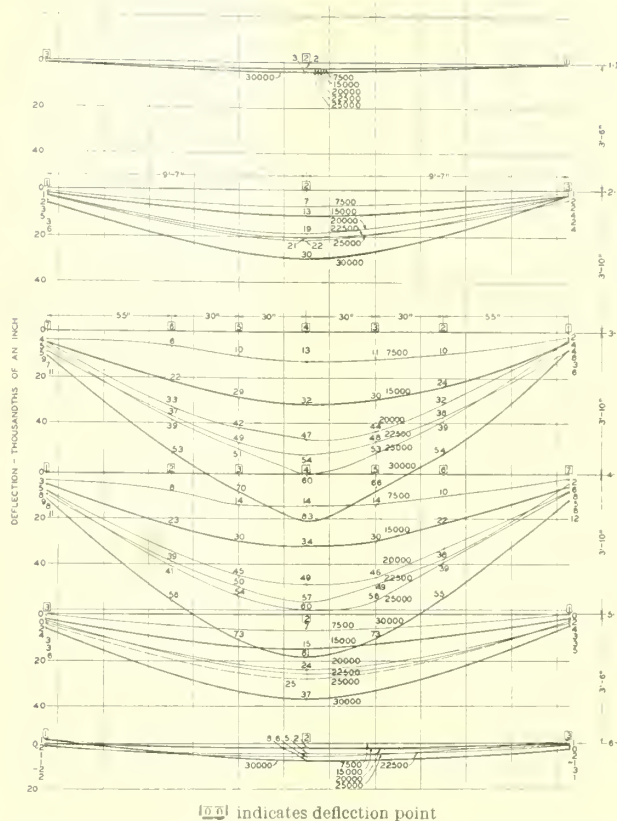


FIG. 9.—DEFLECTION OF I BEAMS FOR 7,500, 15,000, 20,000, 22,500, 25,000 AND 30,000 POUND LOADS APPLIED BEFORE SLAB WAS SUBJECTED TO IMPACT. MEAN VALUES OF DEFLECTIONS SHOWN ON CURVES

machine on the slab. The wheel was then raised free of the slab and the rise of the slab due to the release of sprung load or wheel pressure was measured by pushing the plungers of all dials up against the slab with the finger. The wheel was then dropped and the deflection of the slab due to the impact blow was read on the choked dials.

TEST PROGRAM ON FIRST SLAB

Static loads in increments of 7,500 pounds up to 30,000 pounds were applied at the center of the slab. Deflection readings and strain-gauge records both in the slab and in the steel were made for each increment. The load was removed by decrements of 2,500 pounds and deflection readings at the center of the slab and at the centers of the two center I beams were taken as a measure of the hysteresis or lag of the slab. After an interval of at least 12 hours, to allow complete recovery of the slab, the same series of loads was again applied and similar data taken. This program was repeated and several sets of readings were secured which agreed within close limits. Within this limit of loading no cracks or signs of distress were apparent.

Mean values of deflection are shown in Figures 8 and 9. Deformations in the slab and stresses in the I beams and reinforcing trusses are shown in Figures 10, 11, and 12. Hysteresis or lag is shown in Figures 13 and 14.

The impact machine was then set up with the wheel over the center of the slab (point 51) and a series of blows delivered. These blows simulated one rear wheel of a 5-ton truck carrying a $2\frac{1}{2}$ -ton or a 5-ton load and dropping $\frac{1}{2}$, 1, and $1\frac{1}{2}$ inches. The force of these impact blows was determined by means of the

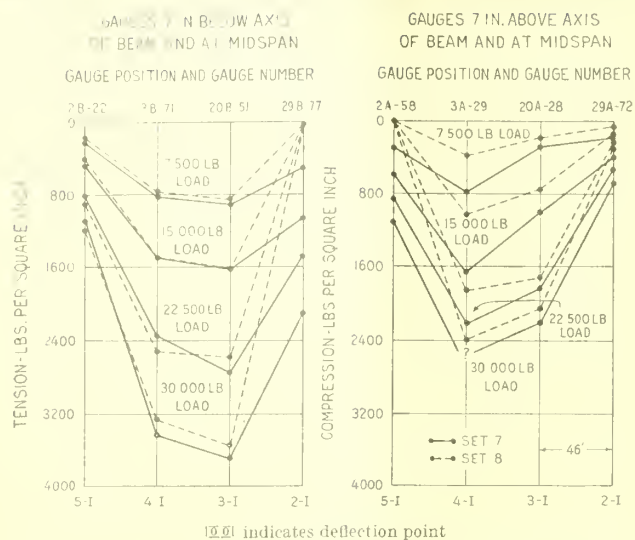


FIG. 10.—INDICATED TENSILE AND COMPRESSIVE STRESSES IN I BEAMS AT GAUGE POINTS AS A RESULT OF STATIC LOADS

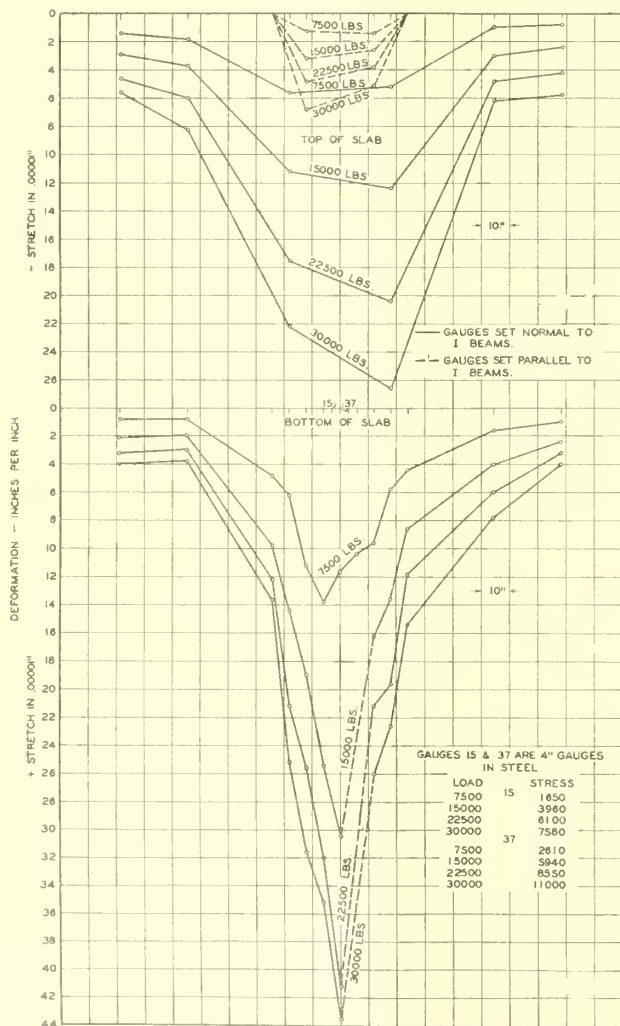


FIG. 11.—DEFORMATIONS IN SLAB ALONG AXIS PARALLEL TO I BEAMS, STATIC LOADS 7,500 POUNDS TO 30,000 POUNDS

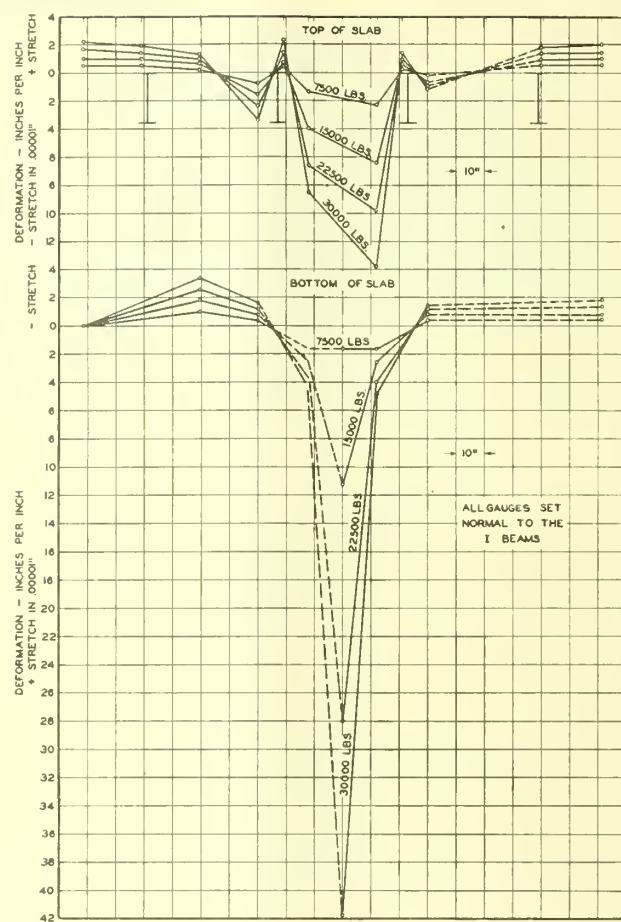


FIG. 12.—DEFORMATIONS IN SLAB ALONG AXIS NORMAL TO I BEAMS. STATIC LOADS 7,500 POUNDS TO 30,000 POUNDS

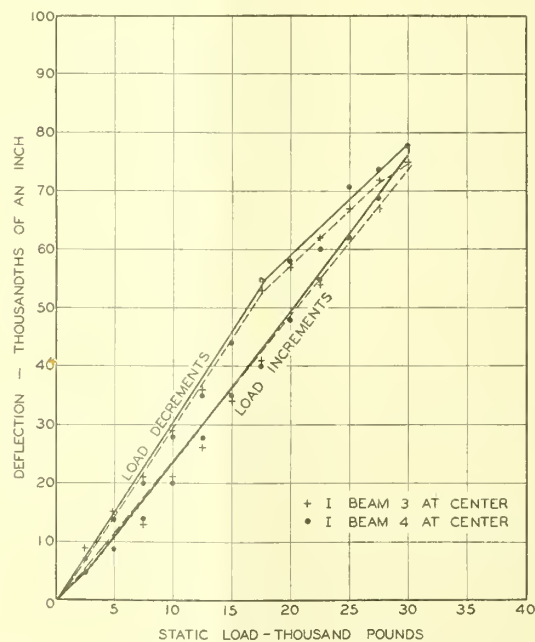


FIG. 13.—HYSTERESIS OR LAG AT MID-SPAN ON I BEAMS 3 AND 4 BEFORE IMPACT

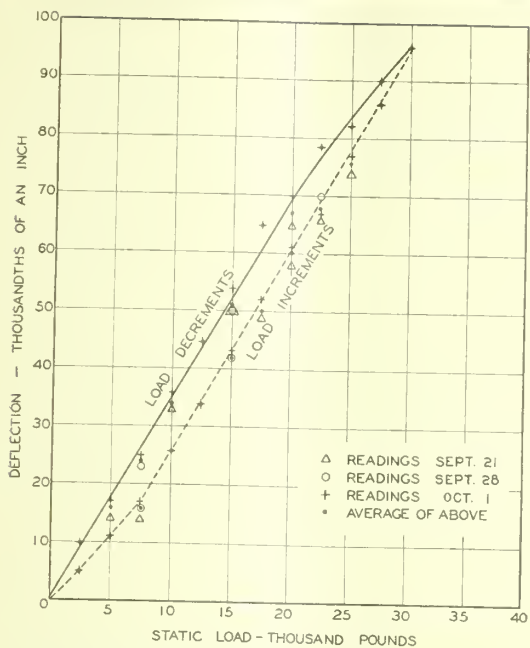


FIG. 14.—HYSTERESIS OR LAG AT CENTER POINT OF SLAB RESULTING FROM STATIC LOADS APPLIED BEFORE IMPACT TESTS

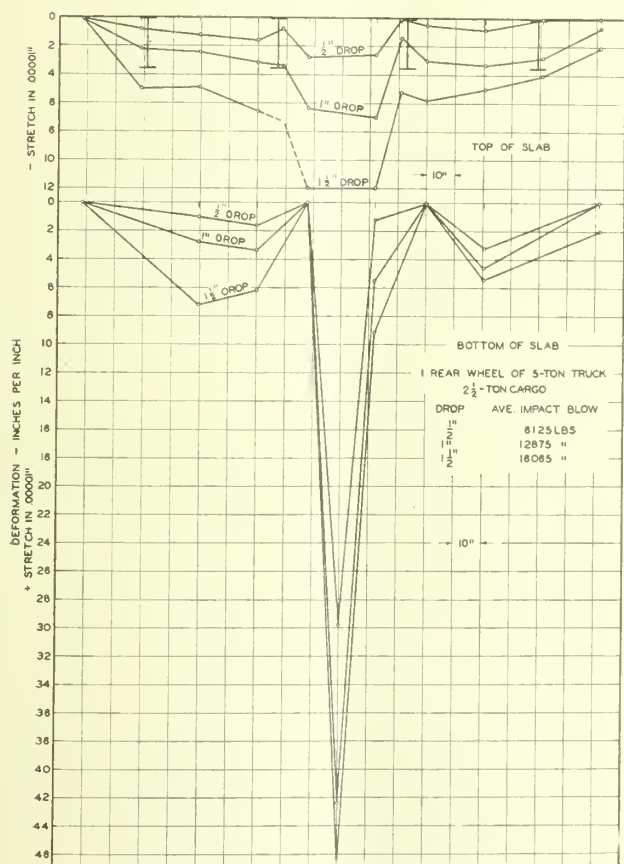


FIG. 15.—DEFORMATIONS IN SLAB ALONG AXIS NORMAL TO I BEAMS DUE TO IMPACT BLOWS OF 5-TON TRUCK WITH 2½-TON LOAD AND DROPS OF ½, 1, AND 1½ INCHES

accelerometer. Deformations in both the top and the bottom of the slab were measured with graphic strain gauges. These data are shown in Table 1 and Figures 15, 16, 17, and 18.

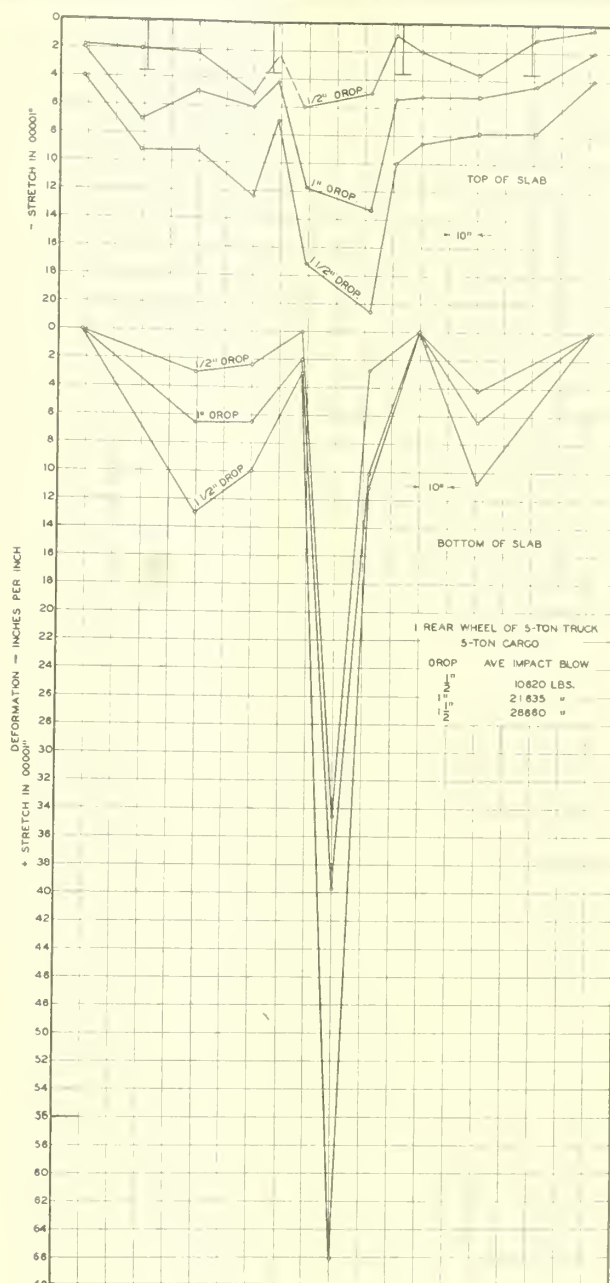


FIG. 16.—DEFORMATIONS IN SLAB ALONG AXIS NORMAL TO I BEAMS DUE TO IMPACT BLOWS OF 5-TON TRUCK WITH 5-TON LOAD AND DROPS OF ½, 1, AND 1½ INCHES

The spring on the impact machine was then adjusted to simulate a 7½-ton truck with a ½-inch drop and a series of 1,000 blows was delivered. Deflection readings were taken at the beginning of the run, at the end of 250 blows and at the end of 1,000 blows as a measure of the fatigue of the slab under continued impact. The impact machine was then removed from the slab and deflections were read as a measure of the permanent set. The results of these measurements are shown in Figure 19. Deflection of the slab under these impact blows in the immediate vicinity of the point of impact was recorded by means of choked dials, and the data are shown graphically in Figure 20.

The impact machine was then moved to a quarter point of the slab centrally between I beams (point 6). A series of 3,000 blows was delivered at this point with a 15,000-pound wheel load and a 5/8-inch drop.

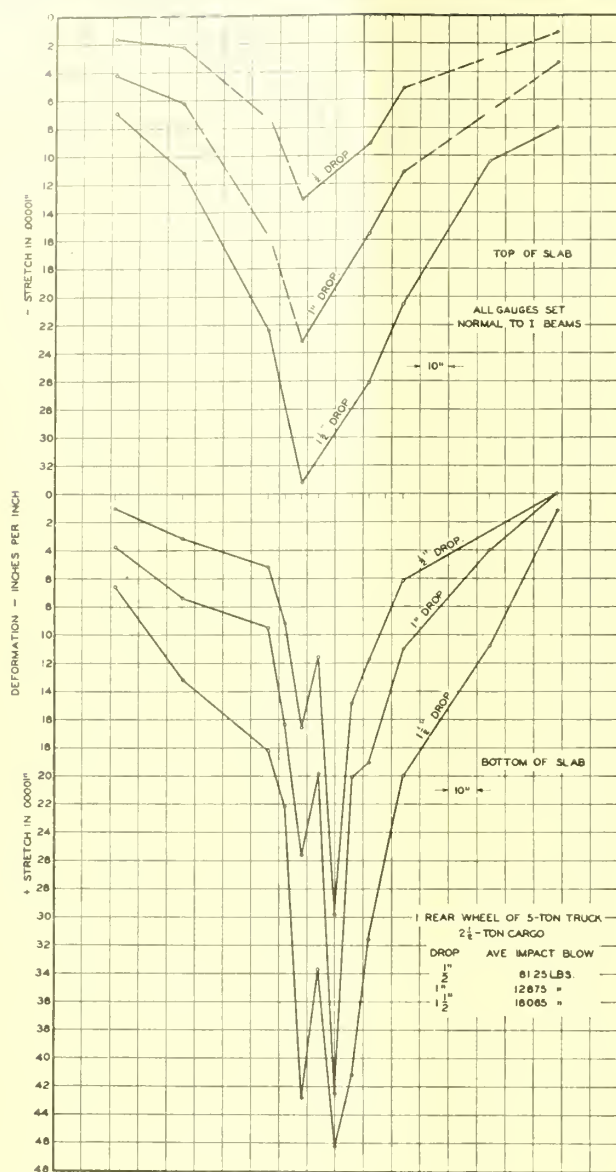


FIG. 17.—DEFORMATIONS IN SLAB ALONG AXIS PARALLEL TO I BEAMS DUE TO IMPACT BLOWS OF 5-TON TRUCK WITH $2\frac{1}{2}$ -TON LOAD AND DROPS OF $\frac{1}{2}$, 1, AND $1\frac{1}{2}$ INCHES

TABLE 1.—Maximum stresses developed in reinforcing trusses under impact at center of slab (graphic strain gauges 15 and 37 set on bottom of two center reinforcing trusses directly below point of impact)

Blow from one rear wheel of 5-ton truck	Drop	Stress in steel in pounds per square inch		Impact blow	Deflection of center of slab	Date
		Gauge 15	Gauge 37			
		Inches	Pounds			
Sprung weight 3,500 pounds (2½-ton cargo); unsprung weight 1,943 pounds	½	2,160	6,185			Nov. 2
	1	5,850	11,640			Do.
	1½	9,060	16,950			Do.
	½	2,340	6,065	0.03		Nov. 2
	1	4,370	5,850	14,110	.07	Do.
	1½	8,250	9,960	19,185	.10	Do.
Sprung weight 5,950 pounds (5-ton cargo); unsprung weight 1,943 pounds	½	3,780	11,115			Nov. 2
	1	8,280	22,255			Do.
	1½	13,410	28,295			Do.
	½	3,630	4,050	10,125	.05	Nov. 2
	1	8,330	9,060	21,015	.10	Do.
	1½	12,350	13,050	29,065	.15	Do.

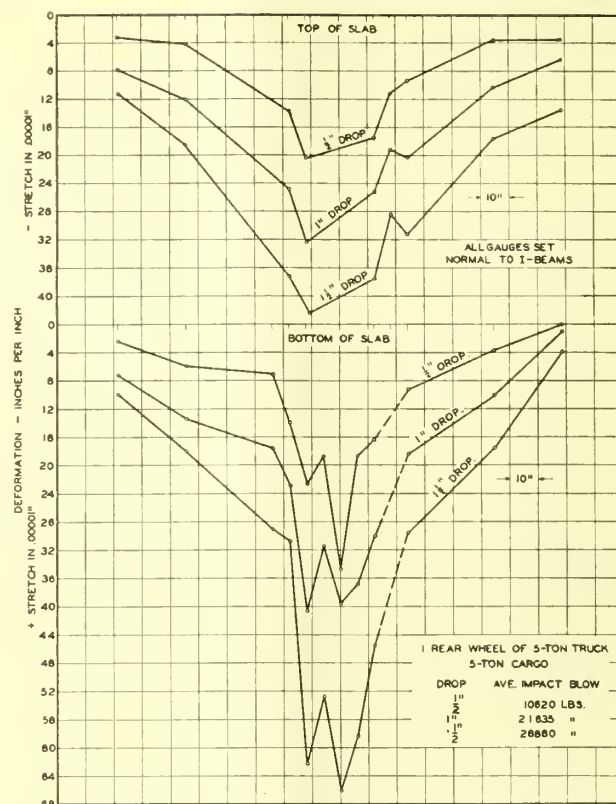


FIG. 18.—DEFORMATIONS IN SLAB ALONG AXIS PARALLEL TO I BEAMS DUE TO IMPACT BLOWS OF 5-TON TRUCK WITH 5-TON LOAD AND DROPS OF $\frac{1}{2}$, 1, AND $1\frac{1}{2}$ INCHES

Graphic strain gauge readings were made on adjacent I-beam stringers, the gauges being 7 inches above and 7 inches below the axes of the beams and opposite the point of impact. Readings were taken at intervals of 500 blows. The data from this test are shown in Table 2.

A series of deflection readings on points 2 to 10 under impact was made at intervals of 500 blows using choked Ames dials. No marked variation in deflections was found during the series of blows. Mean deflections as found are shown in Figure 21.

The attempts made to measure any possible progressive deformation of the slab caused by repeated impact at this point were unsatisfactory. Due to the eccentric position of the point of impact there was a tendency on the part of the slab to continually shift its position on the abutments. Although this movement was small it was sufficient to cause so great a variation between the relative positions of the deflection points on the slab and floor that small changes in the slab could not be measured.

At the end of 3,000 blows very fine hair cracks had developed radially directly under the point of impact. These cracks developed for a length of about 6 inches during the first 300 blows and the remainder during the first 2,500 blows. From then on no further cracking was apparent nor did the existing cracks widen. Sketches of these cracks are shown in Figure 22.

The impact machine was then moved to quarter point 24 and a similar series of 3,000 blows with an average drop of $1\frac{1}{4}$ inches was made. In order to keep the impact machine in place it was necessary to add 2,060 pounds additional weight to the frame, thus increasing its dead weight from 14,580 pounds to 16,640 pounds. Due to the severe impact developed by this drop it

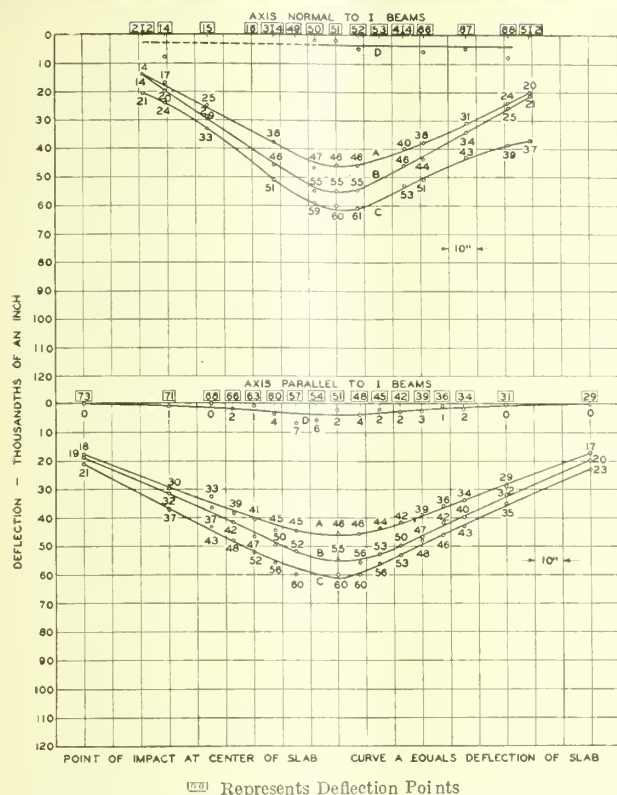


FIG. 19.—EFFECT OF REPEATED IMPACT BLOWS ON DEFLECTION OF SLAB. POINT OF IMPACT AT CENTER OF SLAB. CURVE A REPRESENTS DEFLECTION OF SLAB UNDER WEIGHT OF IMPACT MACHINE + WHEEL LOAD (14,580 + 15,000 POUNDS). CURVE B REPRESENTS DEFLECTION OF SLAB UNDER ABOVE LOAD AFTER 250 BLOWS OF 21,080 POUNDS, WHEEL LOAD 15,000 POUNDS, UNSPRUNG WEIGHT 2,060 POUNDS WITH A $\frac{1}{2}$ -INCH DROP. CURVE C SAME AS CURVE B AFTER 1,000 BLOWS. ON REMOVING MACHINE FROM SLAB, RECOVERY WAS ALMOST COMPLETE. CURVE D REPRESENTS PERMANENT SET AFTER IMPACT MACHINE WAS REMOVED.

was found necessary to guy the impact machine to the I beams at the corners of the slab. The entire slab, including the I beams, crawled diagonally on the abutments as impact blows were applied. During the first 400 blows this movement amounted to $\frac{3}{4}$ inch parallel to the I beams and 1 inch normal to the I beams.

Measurements in this series were restricted to stresses in the I beams adjacent to the point of impact and to

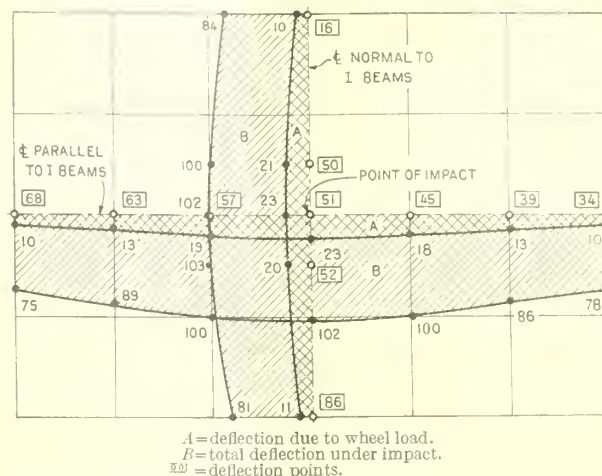


FIG. 20.—DEFLECTION OF SLAB DUE TO IMPACT AT CENTER OF SLAB. UNSPRUNG WEIGHT 2,060 POUNDS. WHEEL LOAD 15,000 POUNDS. AVERAGE IMPACT BLOW 21,080 POUNDS. AVERAGE DROP $\frac{1}{2}$ INCH. FIGURES DENOTE MEAN DEFLECTIONS IN THOUSANDTHS OF AN INCH

deflections under impact. An increase in deflection of the slab under impact was found as the series of blows progressed. The deflection of the slab under impact is shown in Figure 23.

Fine radial hair cracks developed directly under this point of impact as at point 6. These cracks appeared for the most part during the first 600 blows and showed no increase in extent or width at the end of 3,000 blows. A sketch of these is shown in Figure 22.

The entire slab was loosened on the I beams and both slab and I beams had moved two inches at the end of the run. The slab was then jacked back to its original position on the abutment and static loads were again applied at the center in increments of 15,000 pounds up to 75,000 pounds, the safe limit of the loading device. Mean values of several check sets of deflection readings are shown in Figures 24 and 25.

Several sets of graphic strain gauge readings were taken on the two center I beams adjacent to the point of load with results as shown in Figure 26.

The effect of the impact tests on the load-deflection relations of the slab is shown in Figure 27.

Figure 28 shows the deflection set caused by 60,000 and 75,000 pound static loads applied after the impact tests had been completed.

TABLE 2.—Stresses caused by impact in I beams at gauge points

IMPACT AT POINT 24														
At end of blow	4	30	100	300	400	500	600	700	1,000	1,500	2,000	2,500	3,000	Mean
	Pounds	Pounds	Pounds	Pounds	Pounds	Pounds	Pounds	Pounds	Pounds	Pounds	Pounds	Pounds	Pounds	Pounds
Top I beam 2, gauge 41.....	5,160	5,260	6,120	4,460	6,230	6,860	10,910	7,500	7,190	7,300	7,500	7,050	7,560	6,850
Bottom I beam 2, gauge 22.....	8,350	7,970	8,850	9,730	8,560	7,000	7,670	7,350	7,400	7,300	6,770	6,040	7,770	7,760
Top I beam 3, gauge 5.....	5,680	6,070	6,650	5,880	6,510	6,280	8,330	6,500	6,100	6,200	6,250	6,150	5,140	6,290
Bottom I beam 3, gauge 77.....	11,800	15,980	17,080	15,950	18,540	16,900	17,980	13,910	17,400	18,020	13,820	13,330	14,500	15,780

Frame of impact machine weighted with 2,060 pounds. Dead load = 14,580 + 2,060 = 16,640 pounds. Unsprung weight, 2,060 pounds. Wheel load = 15,000 pounds. Stresses read at end of impact blow. Average blow, 40,750 pounds. Average drop, $1\frac{1}{4}$ inches.

IMPACT AT POINT 6								
	1	500	1,000	1,500	2,000	2,500	3,000	Mean
	Pounds	Pounds	Pounds	Pounds	Pounds	Pounds	Pounds	Pounds
Top I beam 2, gauge 41	1,304	2,631	2,331	1,598	2,431	2,465	3,060	2,250
Bottom I beam 2, gauge 22	4,850	3,293	3,782	2,880	3,707	3,734	4,880	3,875
Top I beam 3, gauge 5	3,284	2,970	3,020	2,385	2,650	2,570	3,683	2,940
Bottom I beam 3, gauge 77	2,414	3,338	3,800	3,195	2,948	3,230	5,640	3,510

Dead load = 14,580 pounds. Unsprung weight, 2,060 pounds. Wheel load, 15,000 pounds. Stresses read at end of impact blow. Average blow, 27,030 pounds. Average drop, five-eighths inch.

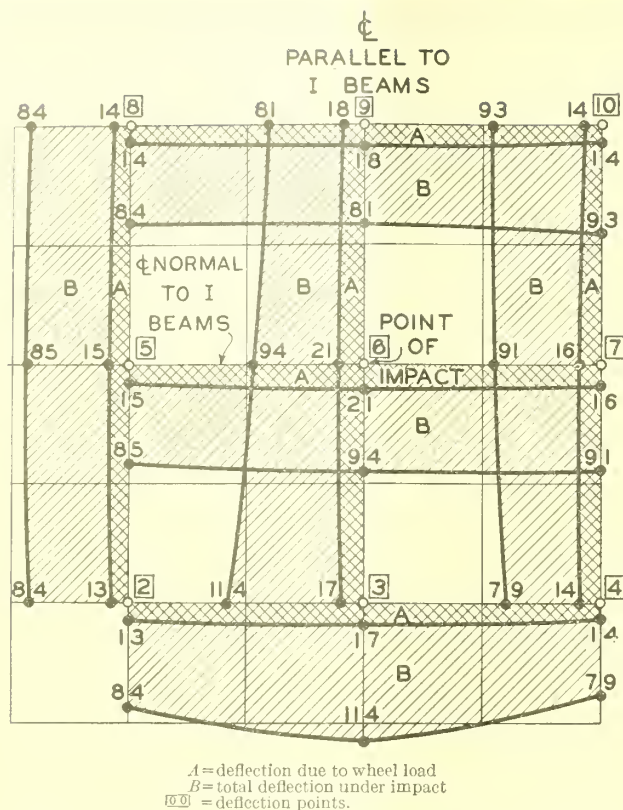


FIG. 21.—DEFLECTION OF SLAB DUE TO IMPACT AT QUARTER POINT No. 6. UNSPRUNG WEIGHT 2,060 POUNDS. WHEEL LOAD 15,000 POUNDS. AVERAGE IMPACT BLOW 27,030 POUNDS. AVERAGE DROP $\frac{5}{8}$ INCH. FIGURES DENOTE MEAN DEFLECTIONS IN THOUSANDTHS OF AN INCH.

BLOWS DELIVERED DIRECTLY OVER POINT 6 3000 BLOWS. $\frac{5}{8}$ " DROP. AVE BLOW 27030*

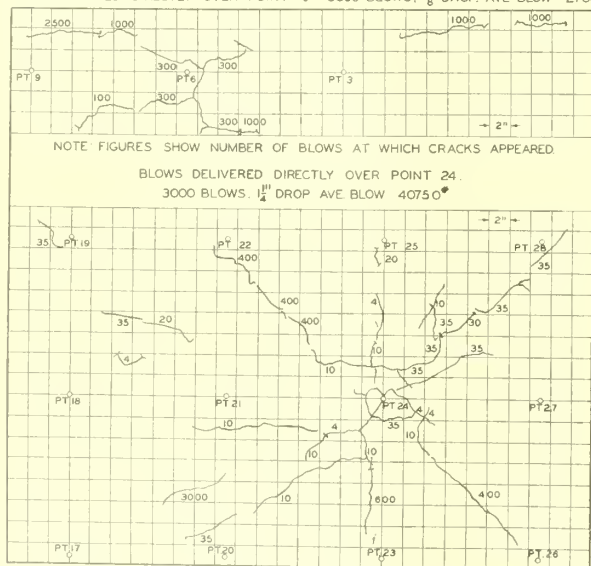


FIG. 22.—SKETCHES SHOWING CRACK DEVELOPMENT AS A RESULT OF IMPACT BLOWS ON POINT 6 AND POINT 24

Check measurements of the spread or tilting of the I beams under 15,000-pound load increments were made as described on page 161. Figure 29 shows that, in spite of the fact that the slab had been loosened from the I beams, the I beams spread or tilted enough to allow their flanges to follow the deflection of the slab.

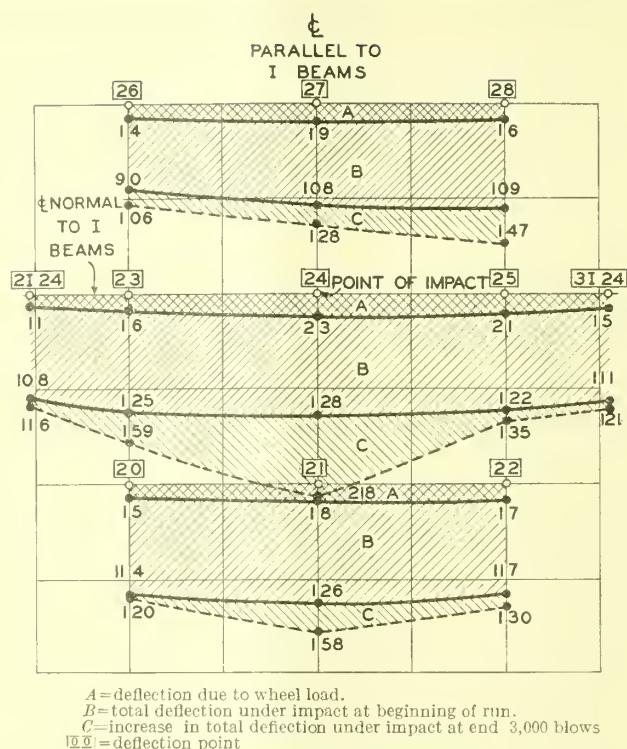


FIG. 23.—DEFLECTION OF SLAB DUE TO IMPACT AT QUARTER POINT 24. UNSPRUNG WEIGHT 2,060 POUNDS. WHEEL LOAD 15,000 POUNDS. AVERAGE IMPACT BLOW 40,750 POUNDS. AVERAGE DROP $1\frac{1}{4}$ INCHES. FIGURES DENOTE MEAN DEFLECTIONS IN THOUSANDTHS OF AN INCH.

Fine hair cracks as shown in Figure 30 developed under static loads of 60,000 to 75,000 pounds. These cracks were very fine and showed no increase in extent or width under repeated loading.

On the removal of the slab one of the reinforcing trusses was broken out of the slab and it was found that in spite of the dry mix used, about 90 per cent of the spaces in the upper chord channels were filled with concrete.

TESTS ON SECOND SLAB

The materials used in the construction of the second slab were identical with those used in the first slab. The cement showed a tensile strength of 300 pounds per square inch and 415 pounds per square inch at the end of 7 and 28 days, respectively. Concrete test cylinders developed a modulus of elasticity of 2,883,000 pounds per square inch and a compressive strength of 3,560 pounds per square inch at the end of 28 days.

The methods of placing and curing were the same in both slabs. The mix was the same, except that the water content was increased, an average slump of 8.7 inches with a 4 by 8 by 12 inch cone, and an average flow of 154 being maintained.

In the tests on the second slab the following measurements were made:

- Deflection of the slab and of the I beams under static loads.
- Deformations in the slab and in the I beams under static loads.
- Spread of two center I beams under static loads.
- Reactions at one end of the I beams.

In testing the second slab measurements were to a great extent confined to one quadrant as it was found

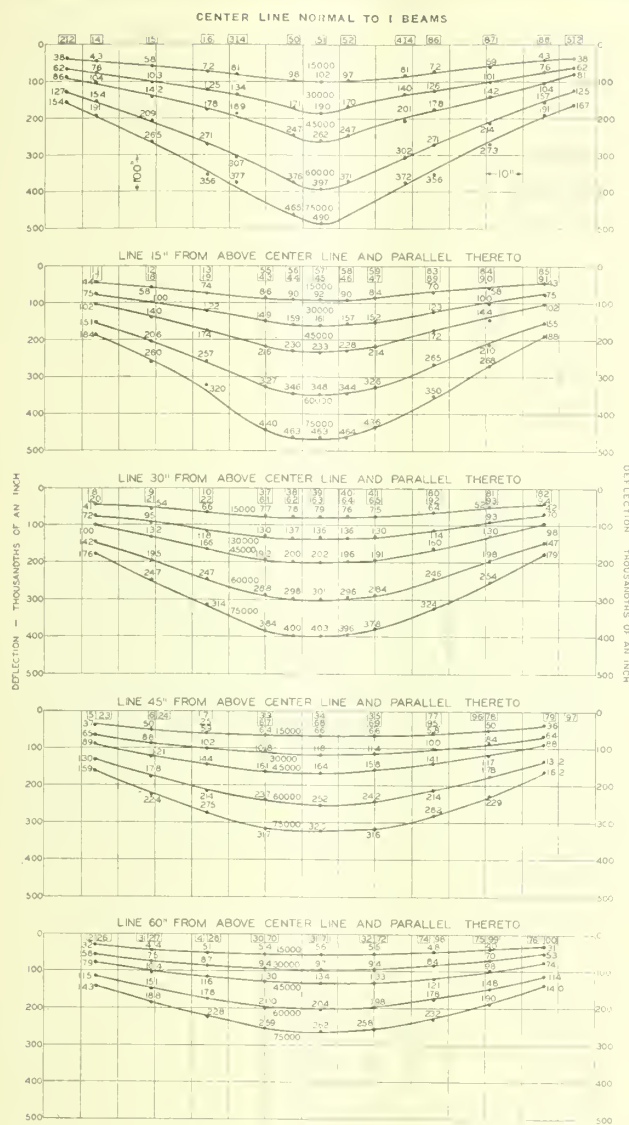


FIG. 24.—SLAB DEFLECTION UNDER INCREMENTS OF STATIC LOADING UP TO 75,000 POUNDS. LOADING APPLIED AFTER IMPACT TESTS

from the first slab that all four quadrants behaved symmetrically.

Static loads were applied at the same point and in the same manner as in the first slab, except that a circular steel plate 12 inches in diameter and $\frac{5}{8}$ inch thick and a cushion of $\frac{3}{8}$ -inch rubber packing were substituted for the cast-iron block and tire sections used on the first slab, thus giving a more definite area of load.

Deflections were measured by wooden rods fitted with metal shoes on the lower end and Ames dials at the top set permanently between the gauge points on the slab and on the I beams and those on the floor. (Fig. 31.) The layout of deflection points and graphic strain gauges is given in Figure 32.

Deformations in the slab and stresses in the I beams were determined by graphic gauges attached as in the previous slab. No stresses were measured in the reinforcing steel in the second slab.

The spread of the two center I beams was measured by the same method as in the first slab, except that two Ames dials were used on each staff, one at either end.

The apparatus used in the measurement of the end reactions of the I-beam stringers consisted of a 2 by

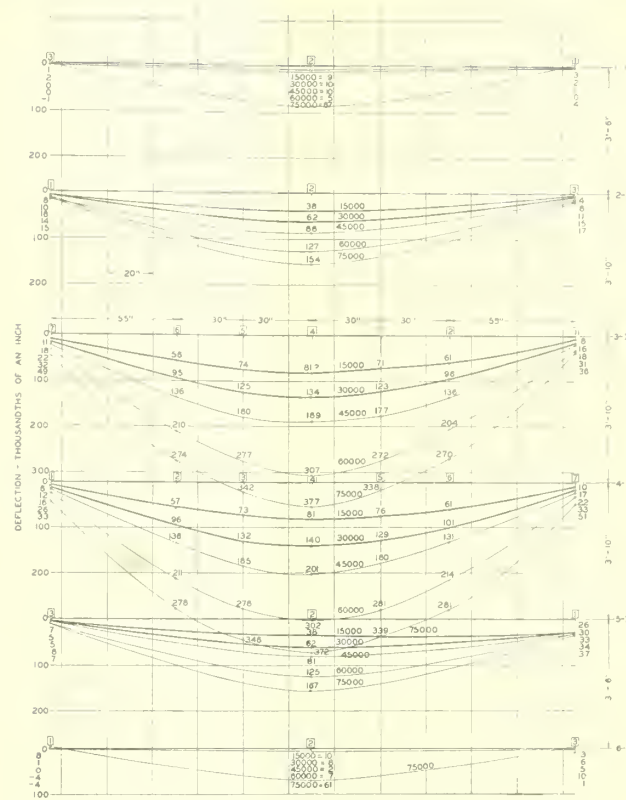


FIG. 25.—DEFLECTION OF I BEAMS UNDER INCREMENTS OF STATIC LOADING UP TO 75,000 POUNDS. LOADING APPLIED AFTER IMPACT TEST

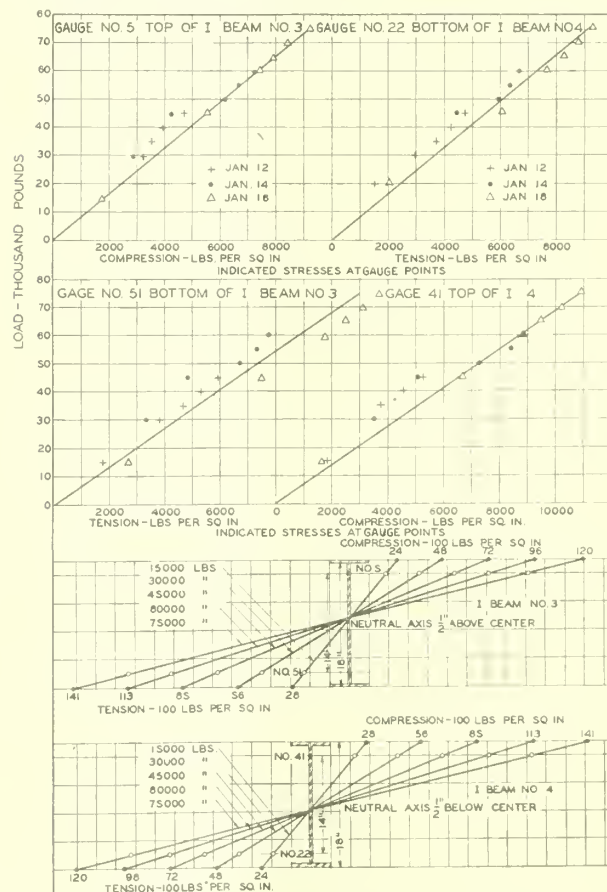


FIG. 26.—POSITION OF NEUTRAL AXIS AT CENTER OF TWO CENTER I BEAMS UNDER STATIC LOAD APPLIED AT CENTER OF SLAB

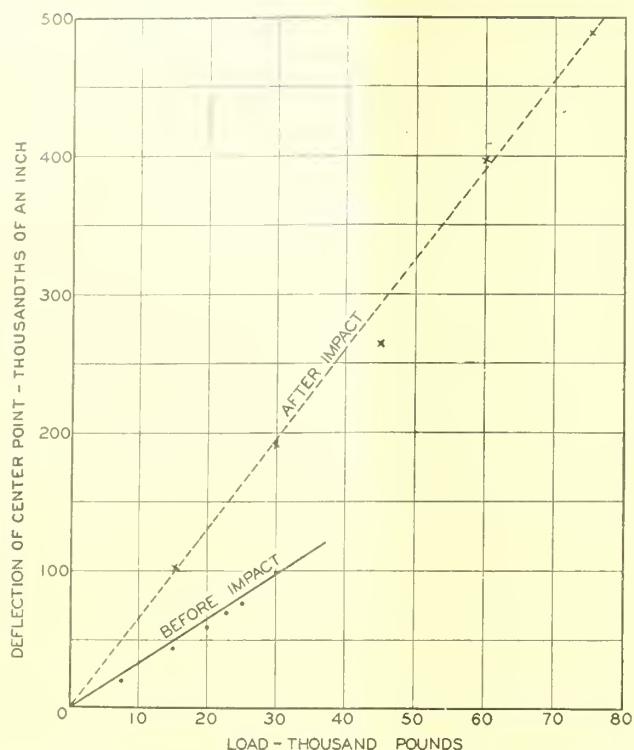


FIG. 27.—DIAGRAM SHOWING LOAD-DEFLECTION RELATION FOR STATIC LOADS APPLIED BEFORE AND AFTER IMPACT TESTS

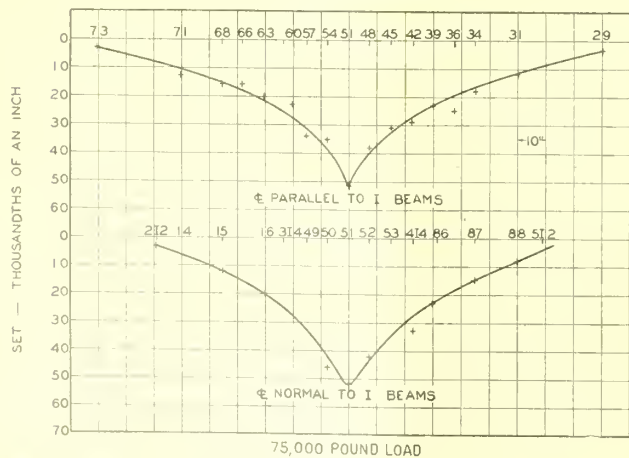
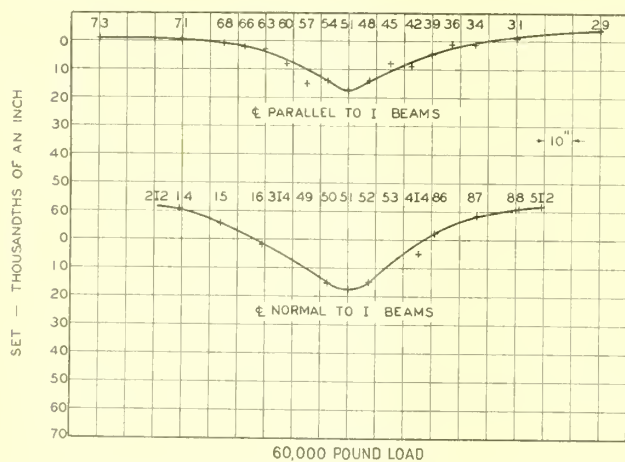


FIG. 28.—DIAGRAMS SHOWING SET CAUSED BY 60,000 AND 75,000 POUND STATIC LOADS APPLIED AFTER IMPACT

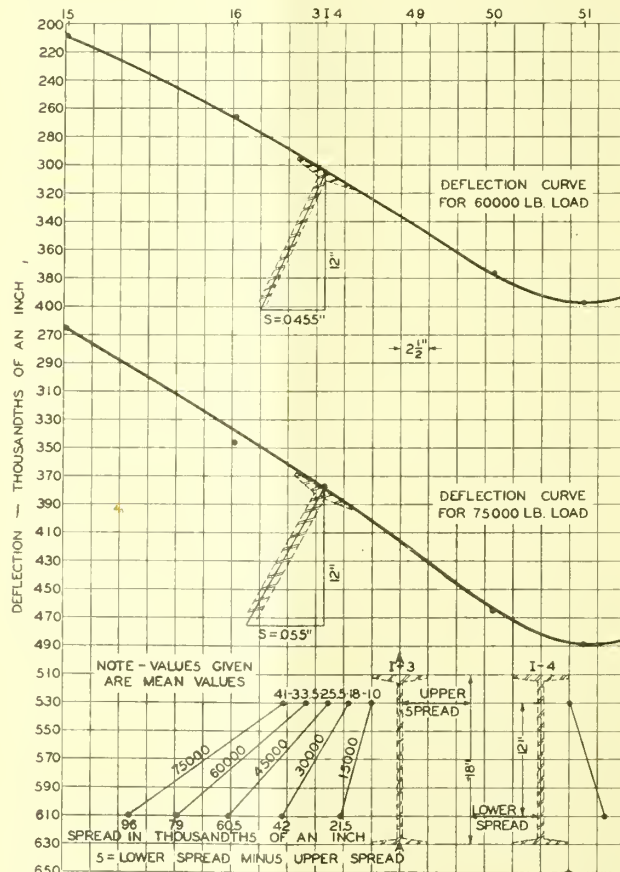
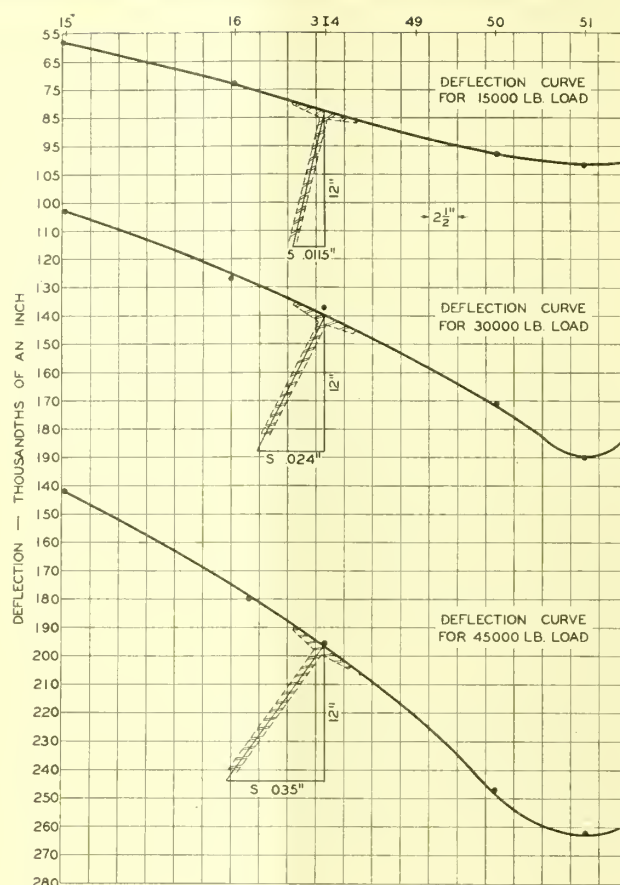


FIG. 29.—SKETCHES SHOWING SPREAD OF THE TWO CENTER I BEAMS UNDER VARIOUS STATIC LOADS

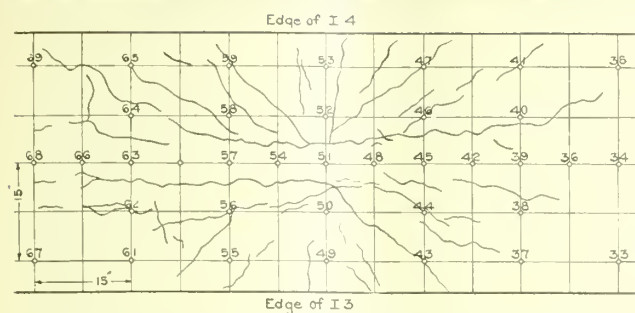


FIG. 30.—SHOWING CRACKS WHICH BECAME VISIBLE UNDER LOADING POINT UPON APPLICATION OF STATIC LOADS FROM 60,000 TO 75,000 POUNDS

4 inch steel beam, 20 feet long, rigidly supported on edge directly above the ends of the I beams by struts resting on the concrete abutment. (Fig. 4.) Six suspension rods of $\frac{5}{8}$ -inch round manganese steel were attached to this beam at their upper ends and to the I beams by suitable stirrups at their lower ends. Nuts at the upper end of these suspension rods permitted them to be shortened, thus taking up the load of the I beams. On opposite sides of the suspension rods gauge points on 15-inch centers permitted the elongation of the rods under load to be measured with a 15-inch Berry strain gauge. Each rod was calibrated and from the mean elongation of the two sets of gauge points the load on each I beam could be determined.

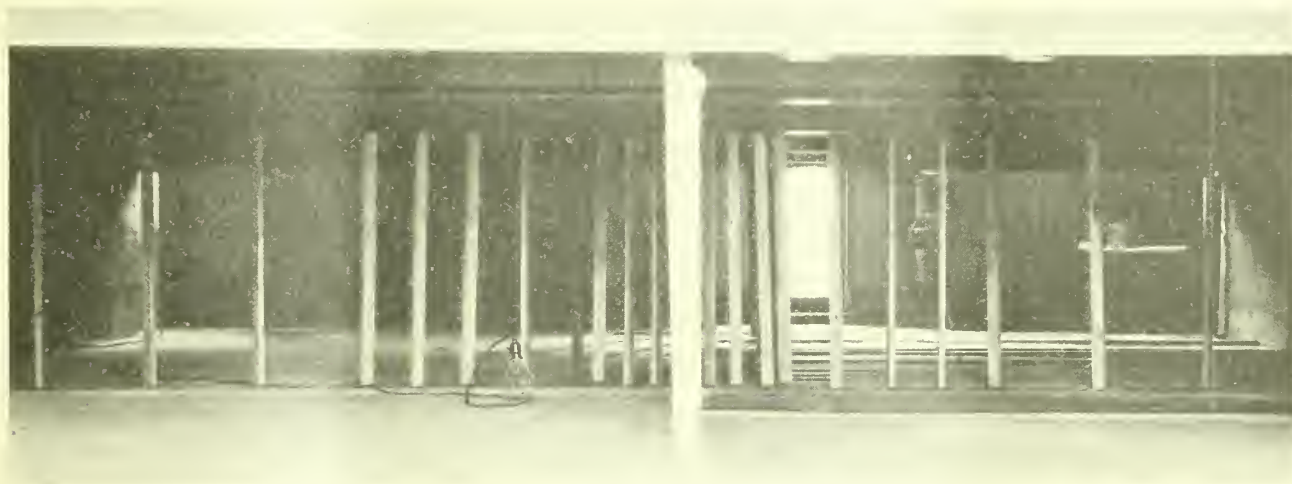


FIG. 31.—WOODEN RODS IN PLACE FOR THE MEASUREMENT OF DEFLECTIONS OF SECOND TEST SLAB

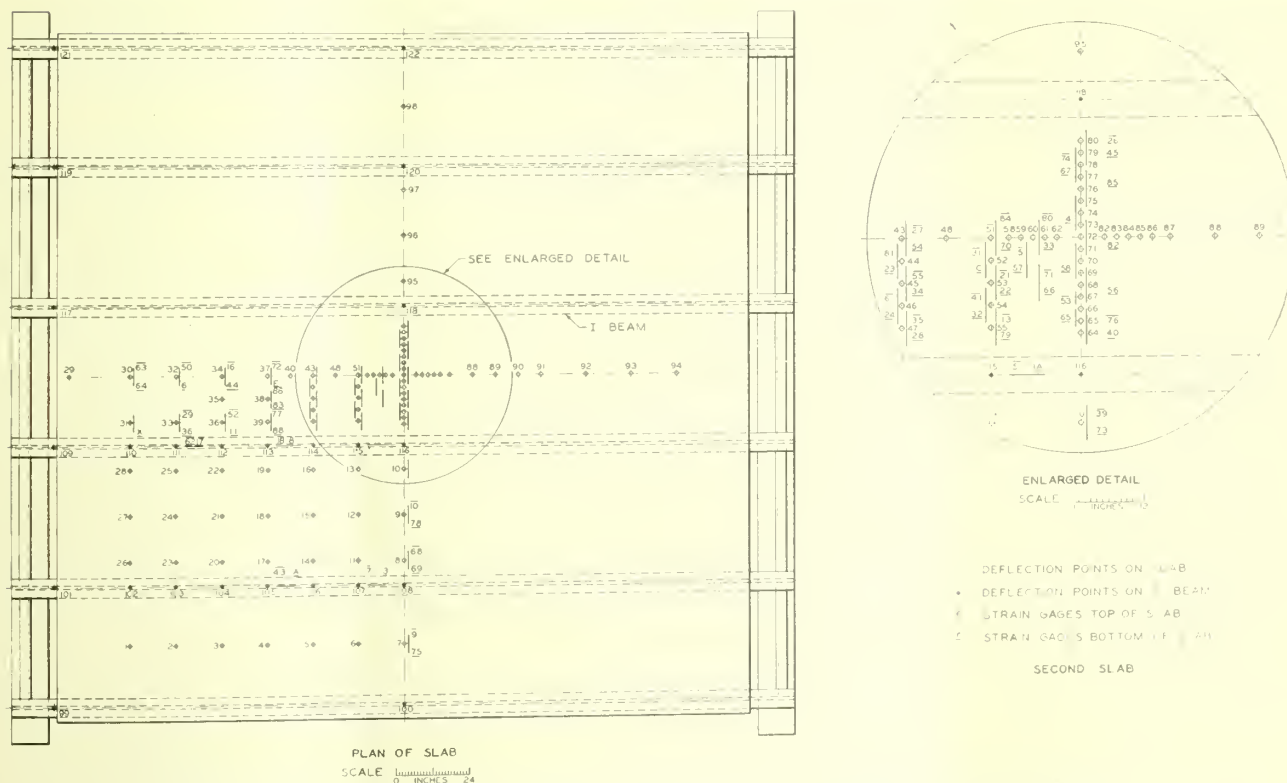


FIG. 32.—LAYOUT OF DEFLECTION POINTS AND GRAPHIC STRAIN GAUGES FOR SECOND SLAB

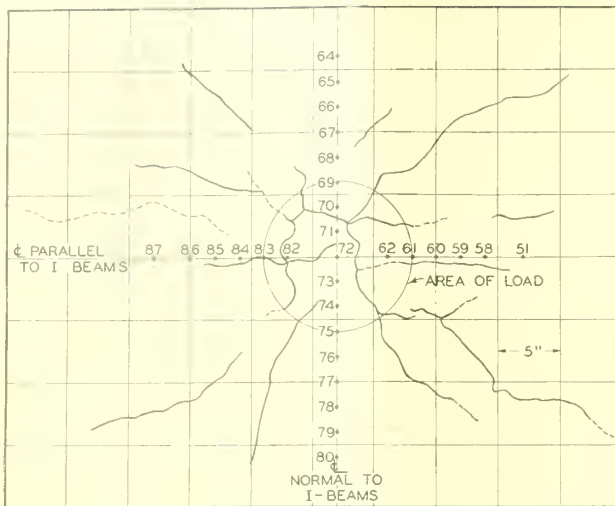


FIG. 33.—SKETCH SHOWING CRACKS DEVELOPED BY STATIC LOADS AT CENTER POINT. SOLID LINES SHOW HAIR-LINE CRACKS DEVELOPED BY LOADS FROM 31,250 TO 40,000 POUNDS. DASH LINES SHOW INCREASE DUE TO 75,000-POUND LOAD

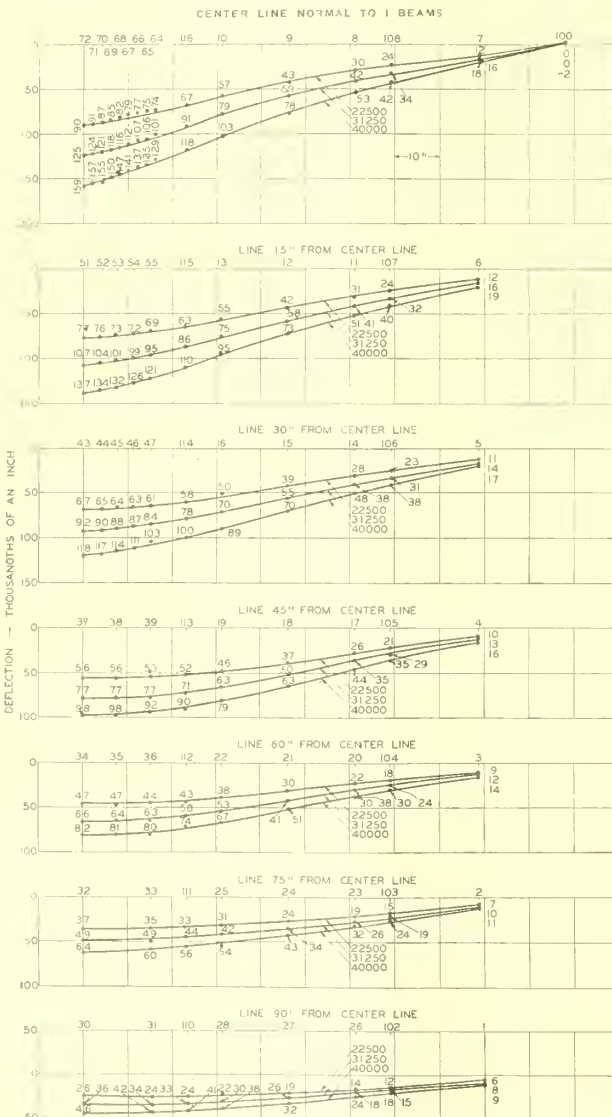


FIG. 34.—DEFLECTION OF SECOND SLAB UNDER DIFFERENT STATIC LOADS APPLIED AT THE CENTER

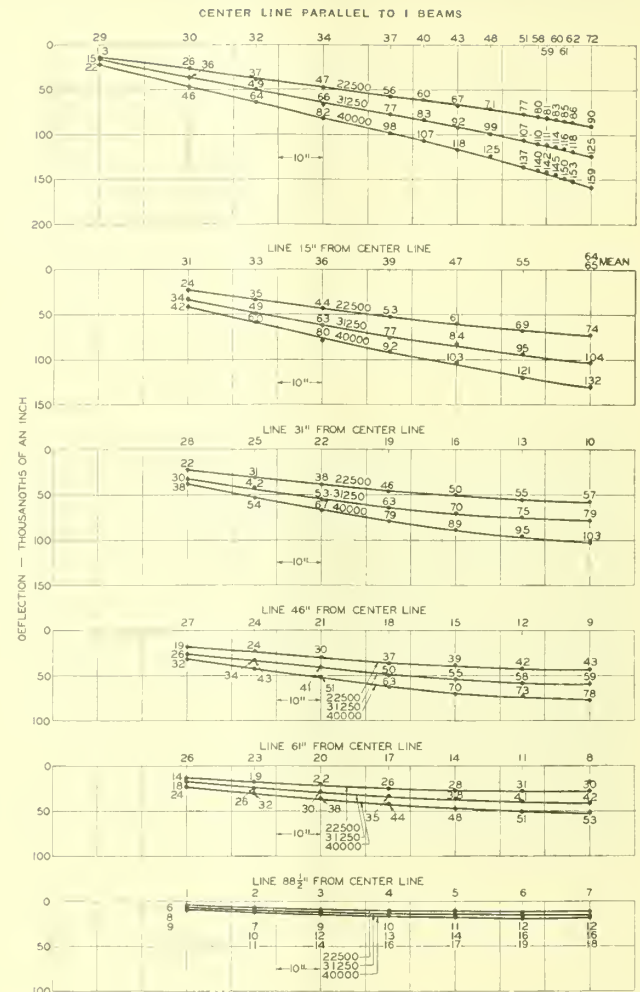


FIG. 35.—DEFLECTIONS OF SECOND SLAB UNDER DIFFERENT STATIC LOADS APPLIED AT THE CENTER

The procedure was as follows:

Wooden rods with Ames dials at their upper ends were set under the deflection points at the ends of I beams nearest the suspension rods. These deflection dials were set to read zero. Several sets of initial readings were taken on each suspension rod with these rods loose. The suspension rods were then shortened sufficiently to allow the bearing plates to be slipped from beneath the I beams and the rods were then adjusted to bring the Ames dials to zero or their original position with bearing plates in place. Strain gauge readings were now taken on each suspension rod as a measure of the dead load of the slab. Static loads were then applied and each time the dials under the ends of the I beams were adjusted by means of the suspension rods to show the same deflection of the I beam that appeared under static load with bearing plates in place. Strain gauge readings were then taken as a measure of the reactions produced by static loads.

PROGRAM FOLLOWED FOR SECOND SLAB

Five check sets of deflection readings were taken for static loads, applied at the center of the slab, of 22,500, 31,250, and 40,000 pounds. Radial cracks as shown in Figure 33 developed under the two larger loads. The mean values of the deflections as measured are shown in Figures 34 and 35.

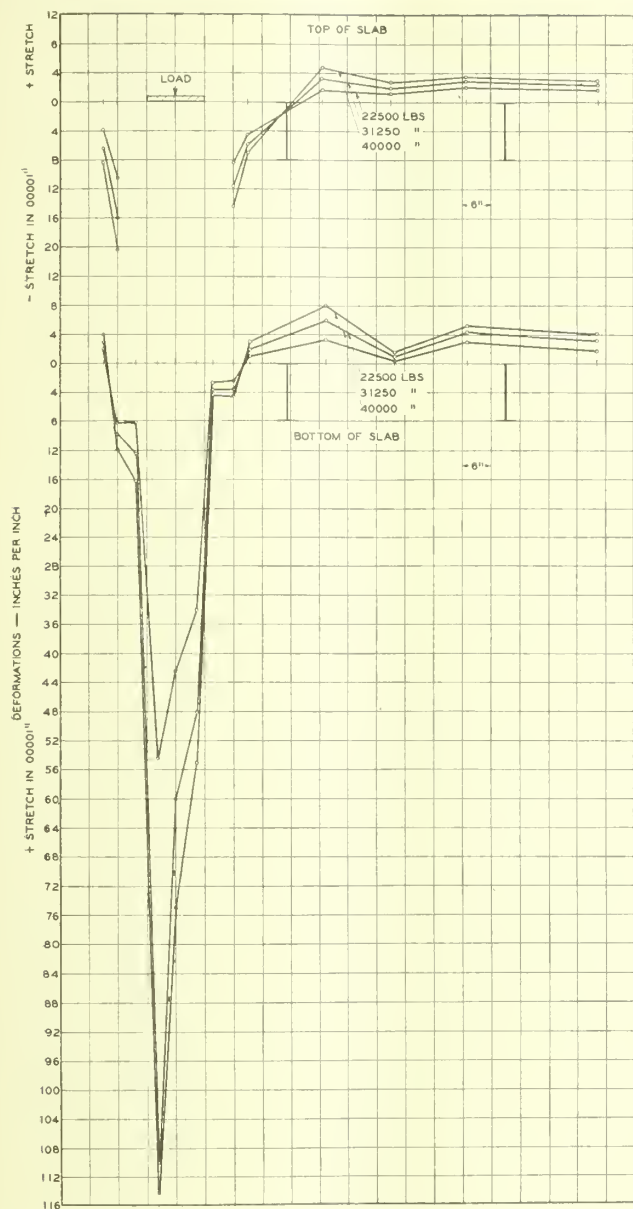


FIG. 36.—DEFORMATIONS IN SLAB ALONG AXIS NORMAL TO I BEAMS UNDER STATIC LOADS OF 22,500, 31,250, AND 40,000 POUNDS

Check sets of graphic strain gauge measurements were made in the top and bottom of the slab. As in all of the strain gauge measurements on this slab the recorded deformations were erratic. It is thought that this is due to local stresses rather than to any fault in the gauges or the technic. When the second slab was built it was necessary to remove bows in the reinforcing trusses by wiring the trusses to the forms. After the concrete set up, this distortion of the trusses must have produced some stress in the concrete which would tend to disturb the measured deformations, particularly under the smaller loads. Deformations are shown in Figure 36.

Six sets of measurements of the spread of the I beams were made under the above static loads with results as shown in Figure 37.

Five sets of reaction measurements at one end of the I beams were then made for each of the above loads. Reactions were also measured for loads from 5,000 pounds to 30,000 pounds by increments of 5,000

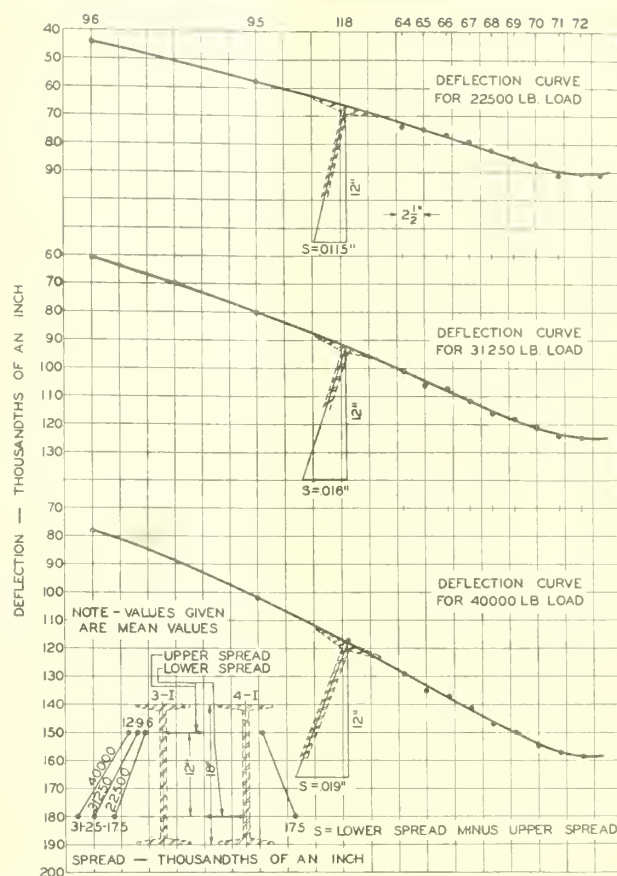


FIG. 37.—SKETCH SHOWING SPREAD OF TWO CENTER I BEAMS UNDER STATIC LOAD DURING TEST OF SECOND SLAB

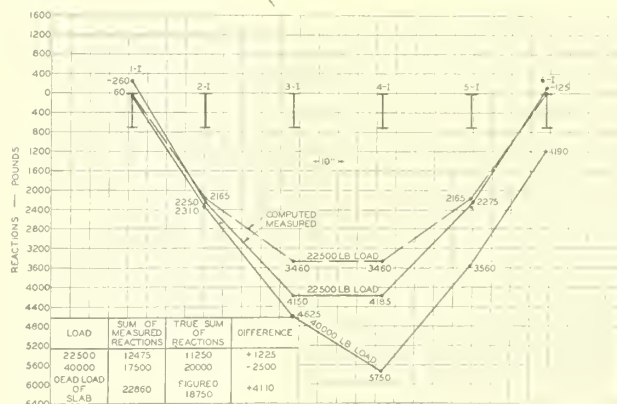


FIG. 38.—I-BEAM REACTIONS AS MEASURED FOR LOADS OF 22,500 AND 40,000 POUNDS. REACTIONS FOR THE 22,500-POUND LOADING ARE THE MEAN OF FIVE SETS OF READINGS AND FOR THE 40,000-POUND LOADING FROM ONE SET. THE COMPUTED REACTIONS ARE DERIVED FROM DEFLECTIONS OF I BEAMS UNDER LOAD WHILE RESTING ON SHIMS

pounds. Results of reaction measurements are shown in Table 3 and Figures 38, 39, and 40.

Check sets of slab deflections were then measured for static loads of 30,000 to 75,000 pounds in 15,000-pound increments with results as shown in Figure 41. Under the 75,000 pound load cracks as developed by the 40,000-pound load were slightly increased, as shown in Figure 33. These cracks were hair cracks and of no serious importance.

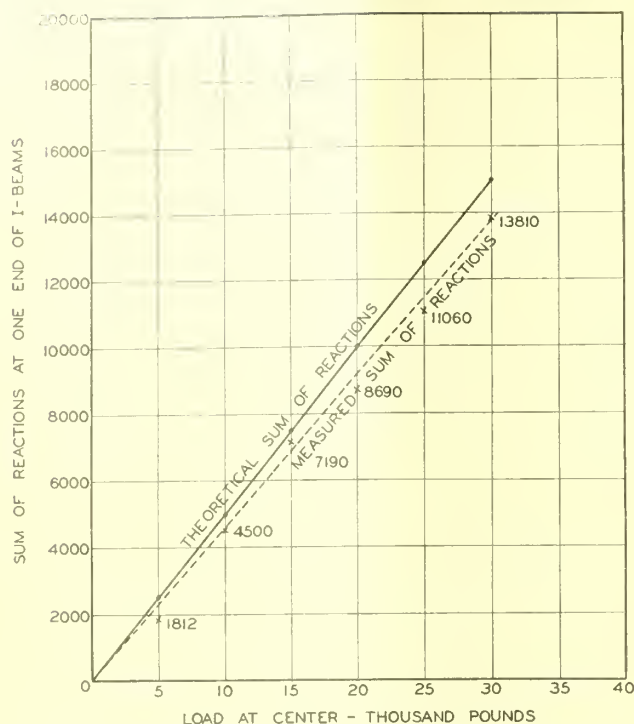


FIG. 39.—VARIATION OF SUM OF MEASURED REACTIONS FROM THEORETICAL VALUES

TABLE 3.—Reactions as measured at I beams for 5,000-pound load increments

[Computed dead load, 37,500 pounds]

Loading	I beam No.						Total	Difference between true and measured reactions
	1	2	3	4	5	6		
Pounds	Pounds	Pounds	Pounds	Pounds	Pounds	Pounds	Pounds	Pounds
Slab only	2,560	4,560	2,500	5,690	125	4,625	20,660	-----
5,000	-310	440	440	875	685	-310	1,820	-680
10,000	-250	1,125	1,060	1,690	1,000	-125	4,500	-500
15,000	-250	1,750	1,560	2,810	1,500	-190	7,190	-310
20,000	-310	1,690	2,560	3,440	1,875	-565	8,690	-1,310
25,000	440	2,125	2,875	4,435	2,440	-375	11,060	-1,440
30,000	-250	2,250	4,190	5,125	2,810	-310	13,810	-1,190

RESULTS INDICATE THAT SLABS ARE ADEQUATE FOR DESIGN LOAD

Regarding these tests purely as a measure of the ability of the floor system to perform safely the duties imposed on it, the following facts are evident.

The design load assumed for the floor slabs was a 15,000-pound static load plus a 50 per cent allowance for impact, or a total load of 22,500 pounds. The first slab under a static load of 30,000 pounds, which is one-third greater than the design load, showed no signs of distress or permanent set.

Under loads of 22,500 pounds and 30,000 pounds the maximum indicated stresses in the reinforcing steel of the first slab directly under the point of load were 8,550 and 11,000 pounds per square inch, respectively, based on the measured value of E of 30,000,000 pounds per square inch.

Referring to Figure 26, it will be seen that for a static load of 30,000 pounds the maximum measured

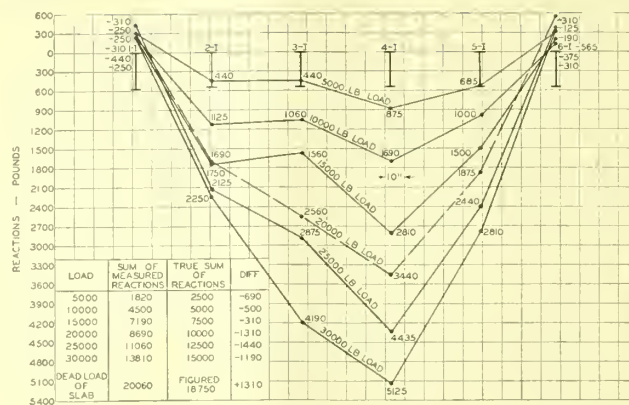


FIG. 40.—I BEAM REACTIONS AS MEASURED FOR 5,000-POUND LOAD INCREMENTS FROM 5,000 TO 30,000 POUNDS

deformations in the I beams indicate a stress of 4,500 pounds per square inch in the steel, at the gauge points, based on a measured value of $E=30,000,000$ pounds per square inch. If a straight line variation of stress across the I beam is assumed, the indicated maximum fiber stress is 5,600 pounds per square inch at the extreme fiber.

Four sets of strain gauge readings in the concrete on the upper surface of the slab, one typical set of which is shown in Figures 11 and 12, indicate that the 22,500 and 30,000 pound loads caused maximum compressive stresses of 470 and 580 pounds per square inch, respectively, along the axis normal to the I beams, while along the axis parallel to the I beams maximum compressive stresses of 930 and 1,210 pounds per square inch were created by the same two loads. These are the maximum stresses indicated in any of the four sets of measurements and not the maxima indicated by the single set shown in Figures 11 and 12.

Slight negative bending moments are indicated in the top of the slab directly over the I beams and along the center line of the slab normal to them, and if it can be assumed that no cracking occurred in the concrete, the deformations measured would indicate tensile stresses in the concrete of 130 pounds per square inch and 160 pounds per square inch for the 22,500-pound and 30,000-pound loads, respectively.

Although deformations equivalent to maximum tensile stresses in the concrete of 1,900 and 2,400-pounds per square inch were found in the bottom of the slab at the center under the above static loading the reinforcing steel was ample and no visible cracks developed. The loadings referred to were static.

Fatigue due to the dead load of the impact machine of 14,580 pounds plus repeated impact blows of about 21,000 pounds was apparent but recovery was almost complete after an interval of rest.

Under an impact blow of about 29,000 pounds a maximum stress of somewhat over 13,000 pounds per square inch was developed in the reinforcing trusses directly under the blow. For the same blow a maximum compression in the slab of 855 pounds per square inch along the axis normal to the I beams and 1,764 pounds per square inch along the axis parallel to the I beams was found.

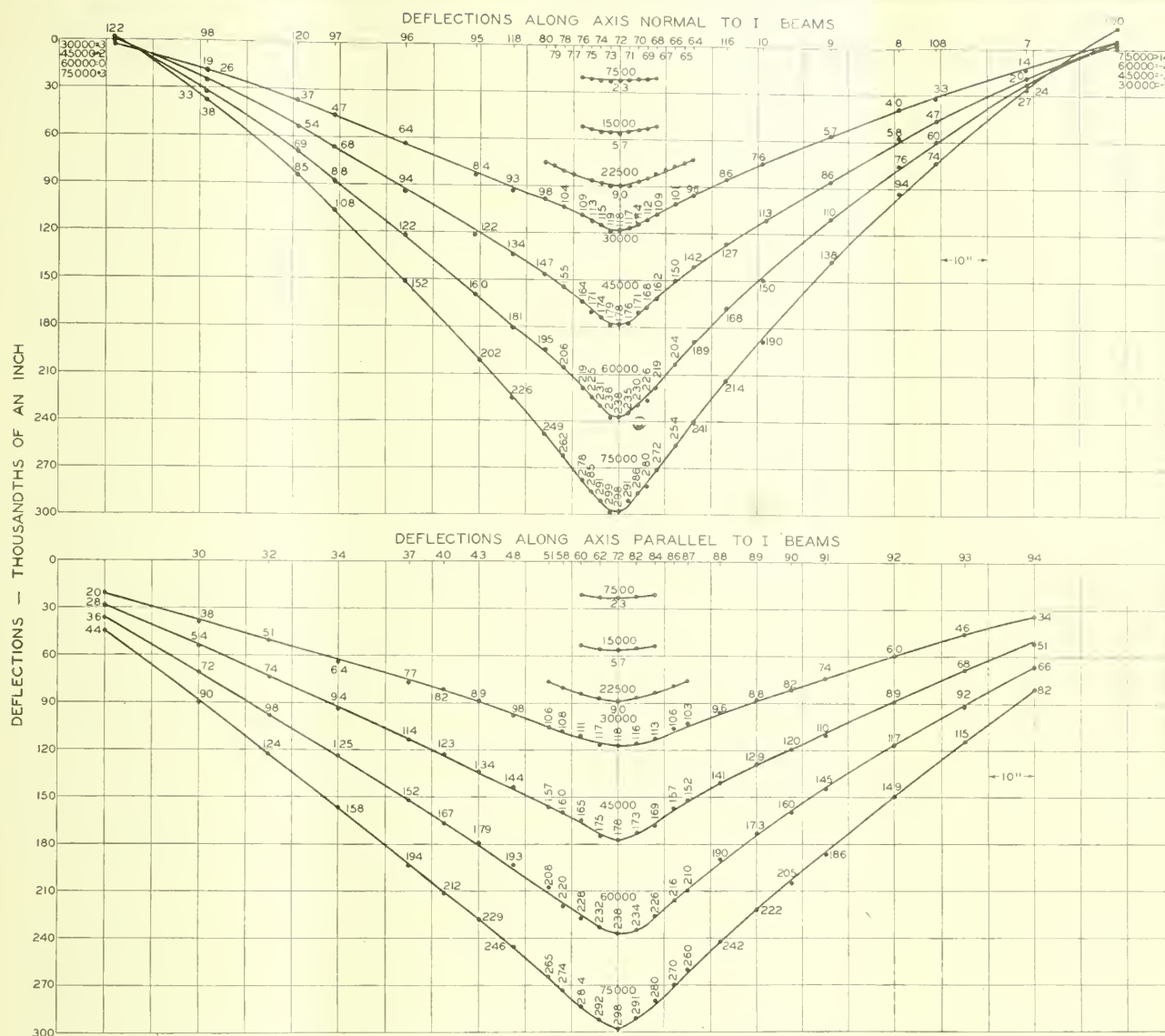


FIG. 41.—DEFLECTION OF SECOND SLAB UNDER STATIC LOADINGS FROM 30,000 TO 75,000 POUNDS. VALUES USED ARE MEAN DEFLECTIONS

Deformations equivalent to a maximum tensile stress in the concrete of 2,754 pounds per square inch were found in the bottom of the slab under the 29,000-pound impact pressure, yet no visible cracks appeared.

A series of 3,000 impact blows of about 27,000 pounds intensity at a quarter point on the slab developed a maximum mean indicated stress at the gauge points of 3,875 pounds per square inch in the *I* beams, while a series of 3,000 blows of about 41,000 pounds intensity at a second quarter point resulted in a mean indicated stress at the gauge points, of 15,780 pounds per square inch in the *I* beams. (See Table 2.)

Some few hair cracks developed directly under the point of impact at both quarter points, but as these cracks developed during the early part of each series of blows, were not progressive and showed practically no increase at the end of the run, they are not to be considered as of serious importance. Although the first slab showed a marked increase in flexibility due to continued impact at the center no signs of distress were apparent until a static load of 60,000 pounds had been applied. Under loads from 60,000 to 75,000 pounds, hair cracks similar in type to those at the

quarter points developed on a limited area directly under the point of load. It is probable that the increase in flexibility is due, in part to fatigue, and in part to a decrease in the T-action of the slab and *I* beams, caused by a breaking of the bond between the slab and the *I* beams by the series of severe impact blows.

The increased water content used in the second slab caused a decrease in compressive strength of 35 per cent (from 5,465 pounds to 3,560 pounds per square inch) and under the 30,000 pound load an increase in deflections or flexibility of about 18 per cent with a corresponding increase in deformations and stresses. This slab under a static load of 31,250 pounds evidenced no signs of distress. At some point between loads of 31,250 pounds and 40,000 pounds a few fine hair cracks developed directly under the load, but an increase of load to 75,000 pounds caused very slight increases in these cracks, which may be taken as evidence that the load was well taken care of by the reinforcing steel. As the concrete actually used on the bridge floor slabs showed a mean compressive strength of 2,700 pounds per square inch probably due to its greater water content, increased deflections and stresses over those found in

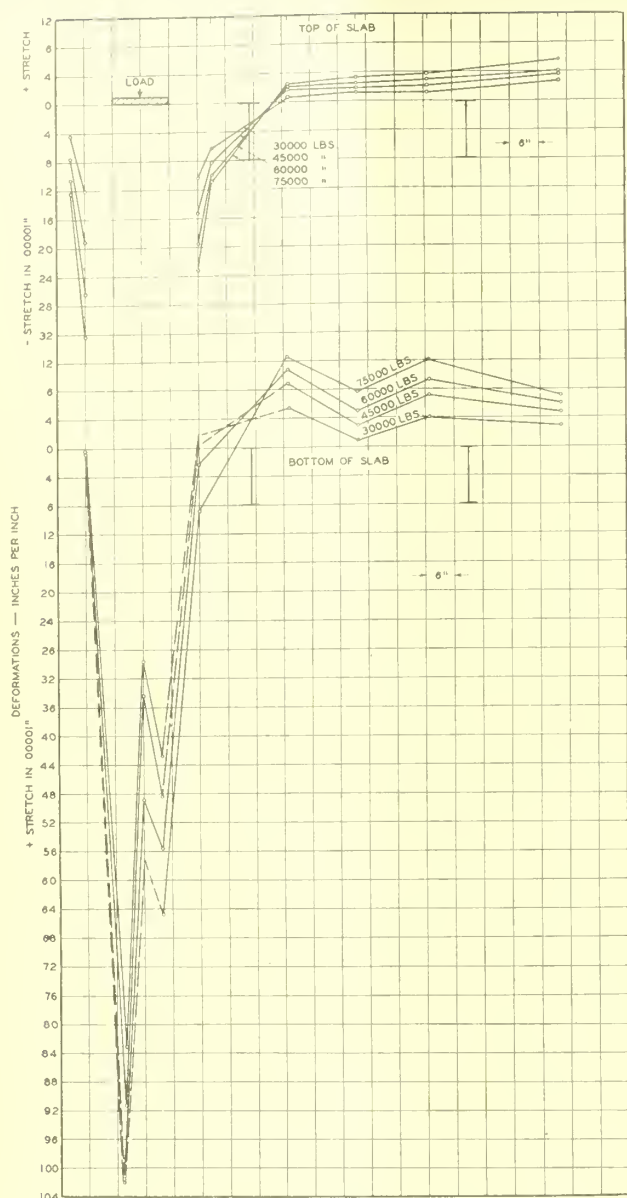


FIG. 42.—MEAN DEFORMATIONS IN SLAB ALONG AXIS NORMAL TO I BEAMS FOR STATIC LOADS OF 30,000 POUNDS TO 75,000 POUNDS

either of the test slabs are to be expected, but as the factors of safety are large enough to care for such an increase it is quite apparent that the slabs are ample to withstand any possible loading to which they may be subjected by modern traffic.

RESULTS CONSIDERED AS A POSSIBLE AID TO DESIGN

As the data collected during these tests will undoubtedly be of value to those who are interested in developing a method of design, they are presented as fully as possible in this report, with a discussion of their relative value and probable accuracy.

It is felt that of the measurements made in these tests, the deflection measurements are the most reliable because:

1. The mechanical methods used in making them allow but small chance for physical errors to creep in.
2. Repeated sets of readings on both slabs give close checks for the individual slabs.
3. Deflection readings for both the first and second slab check each other closely as to type of deflection curve.

It will be seen from Figures 8, 24, 34, 35, and 41 that both slabs deflect symmetrically about their axes and that deflections vary directly with the static loads.

From Figures 9 and 25 it appears that the I-beam stringers also deflect symmetrically, and with the slab, as deflections of the I beams coincide with the deflection curves of the slabs. Figures 29 and 37 clearly show that the flanges of the I beams tilt and follow the curve of the slab, the entire slab deflecting as a simple plate being slightly stiffer along the axis normal to the I beams. The above action is quite evident from Figure 43.

The first slab under static loads up to 30,000 pounds showed no permanent set, although the expected lag was observed in both slab and steel, as shown in Figures 13 and 14.

Progressive fatigue with increased flexibility under continued impact blows delivered at short intervals with almost complete recovery after an interval of rest is evident from Figure 19. From Figures 8, 24, and 27 it will be seen that deflections of the slab under equal static loads have been almost doubled by the fatigue caused by 7,000 impact blows and decrease in T-beam action of the I beams and slab acting as a unit.

Figures 8 and 24 indicate that the type of deflection curve after impact is the same as that before impact, varying only in amount. This is more clearly shown in Figure 44, where deflection curves for the first slab both before and after impact and for the second slab are reduced to type curves. Deflection curves for the first slab under a static load of 30,000 pounds before impact were taken as base curves. Curves for the first slab under static load after impact and the second slab under static load were reduced thereto by applying factors equal to the ratios of the deflections at the center points under 30,000 pound loads to the center deflections of the base curve.

It appears from these type curves that the above three cases do not vary in type but vary only in the amount of the deflections, due probably to fatigue and increased flexibility in the case of the first slab after impact and to the weaker concrete in the second slab. Therefore it may be assumed that the action of the slabs in all three cases was identical.

In Figure 45 a comparison of the deflections of the first slab under static load both before and after impact with the deflections as measured by "choked" dials under an impact blow equal to the static load is shown. This comparison shows the progressive fatigue due to the impact and also indicates that the type of the deflection curves is essentially the same for both static loads and impact blows of equal magnitude.

In Figure 46 a comparison is made of the stresses in the reinforcing trusses under impact and static pressures, by drawing the curve which shows the static load-stress relation (data from Figure 10) and then plotting the impact data shown in Table 1 against this curve. The indication is that somewhat higher stresses result from impact pressures than do from static pressures of the same magnitude. The stresses under static load appear to vary directly with the load. This is also true of the static load stresses in the I beams as shown in Figure 26.

Stresses as measured in the concrete of the two slabs were more or less erratic and unsatisfactory. The reinforcing trusses on account of their weight, length, and method of fabrication were warped in both planes,

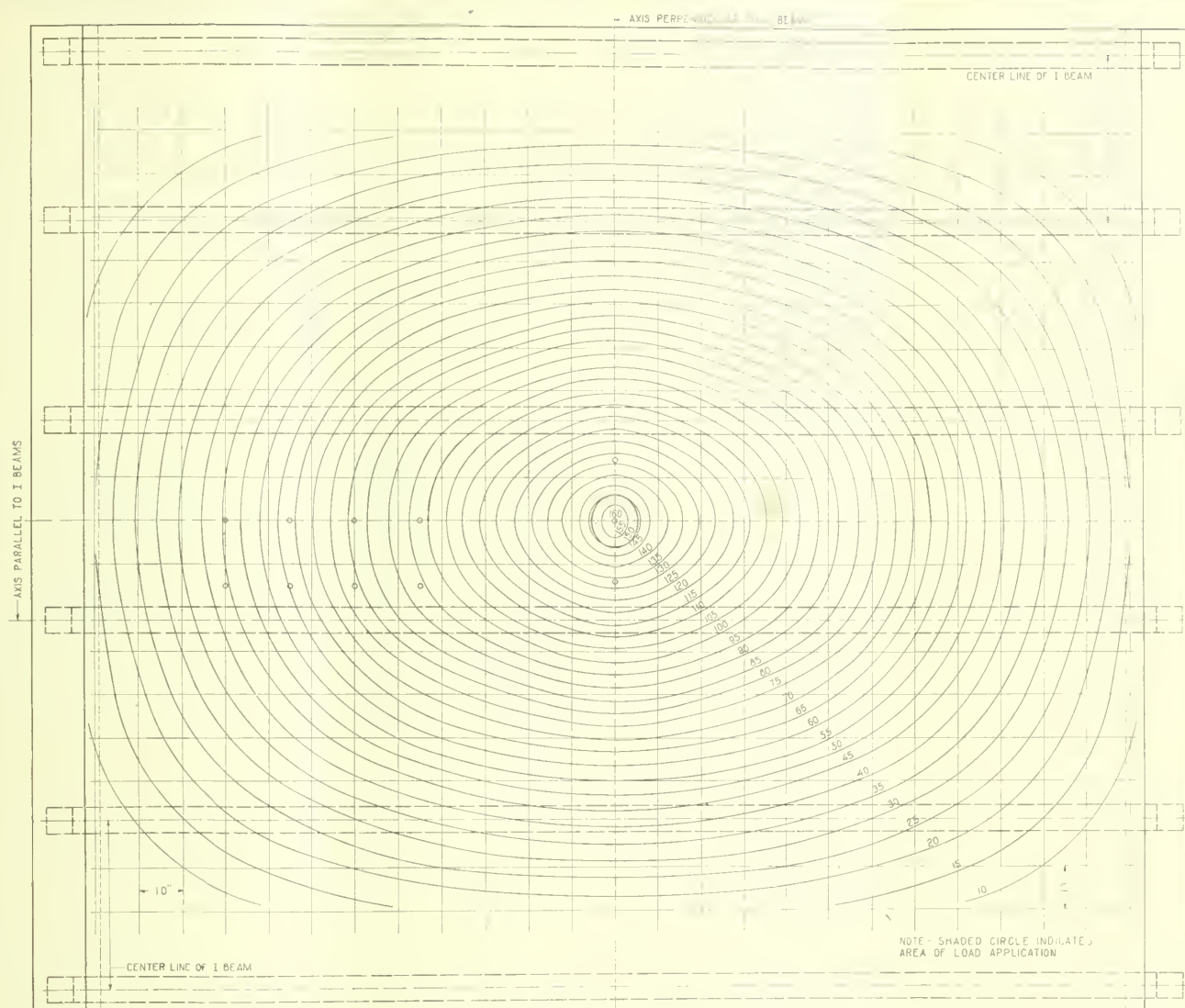


FIG. 43.—SKETCH SHOWING DEFLECTION CONTOURS PRODUCED BY A CENTER LOAD OF 40,000 POUNDS, CONTOUR INTERVAL 0.005 INCH

and, as previously remarked, it was necessary to spring them into place and wire them to the forms. On removing the forms there is little doubt that the spring action of these trusses introduced initial stresses in the slab in some cases large enough to partially neutralize or appreciably increase the stresses caused by static loads and impact.

Difficulty was experienced in getting satisfactory readings on all of the large number of gauges used in these tests. It was necessary to allow these gauges to remain in place for comparatively long periods which may have introduced slight errors due to temperature changes or dust on the recording slides.

Tests of the graphic strain gauges, under impact, made at Johns Hopkins University have shown them free from inertia effects and their records to be reliable. Nevertheless, difficulty was experienced in keeping gauge points in the slab from loosening and the glass slides from slipping, especially on the bottom of the slab, when delivering severe impact blows.

The principal value of these measurements is to show the approximate maximum stresses developed under the loads imposed.

Measurements of reactions are shown in Figures 38 and 40. In Figure 39 the variation between the sum of the measured reactions and the load imposed varies from 4 to 28 per cent, the 28 per cent variation being for the low load of 5,000 pounds.

The reactions shown seem to support the contention that there was present a certain arching action in the slab due to the spring action of the steel. It is to be noted that as the load increased the reactions more nearly approach symmetry and the error decreases to a reasonable variation of about 10 per cent. By a comparison of results as shown in Figures 38 and 40 it will be seen that for I beams 2, 3, 4 and 5, reactions as measured vary directly as the loads applied within the limits of accuracy of these measurements. The measurements at I beams 1 and 6 are so small as to be indeterminate considering the methods used in making the measurements.

A SUGGESTED METHOD OF ANALYSIS

No attempt to develop a possible method of analysis for this type of slab has been made by the Bureau of Public Roads. However, engineers of the Delaware

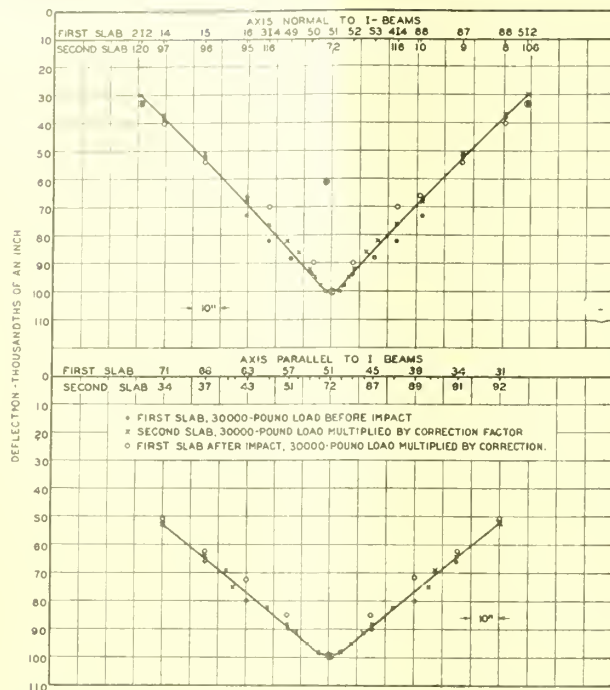


FIG. 44.—COMPARISON OF TYPES OF DEFLECTION CURVES FOR FIRST AND SECOND SLABS. CORRECTION FACTORS FOR DEFLECTION OF FIRST SLAB UNDER STATIC LOADING AFTER IMPACT TESTS AND FOR SECOND SLAB DETERMINED AS EXPLAINED IN TEXT

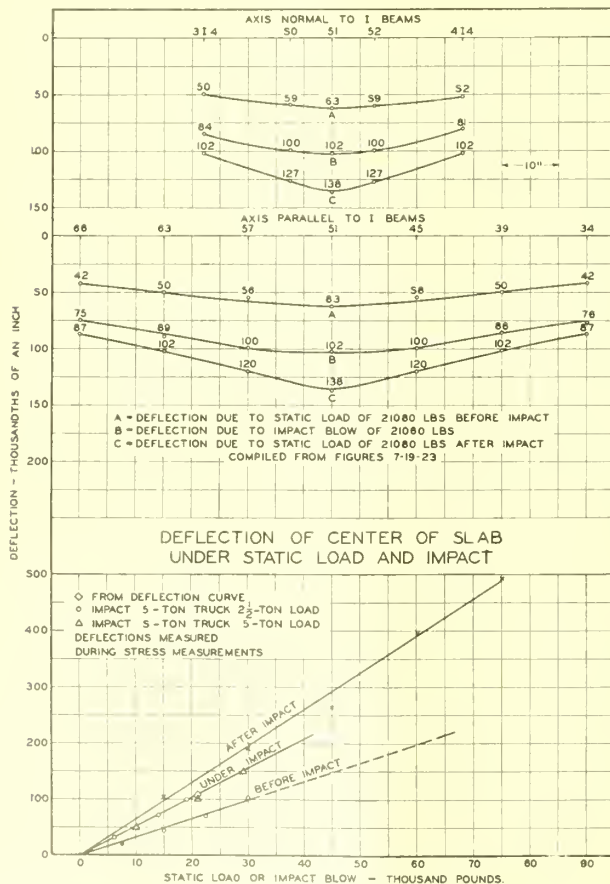


FIG. 45.—COMPARISON OF DEFLECTION OF SLAB UNDER STATIC LOAD AND IMPACT

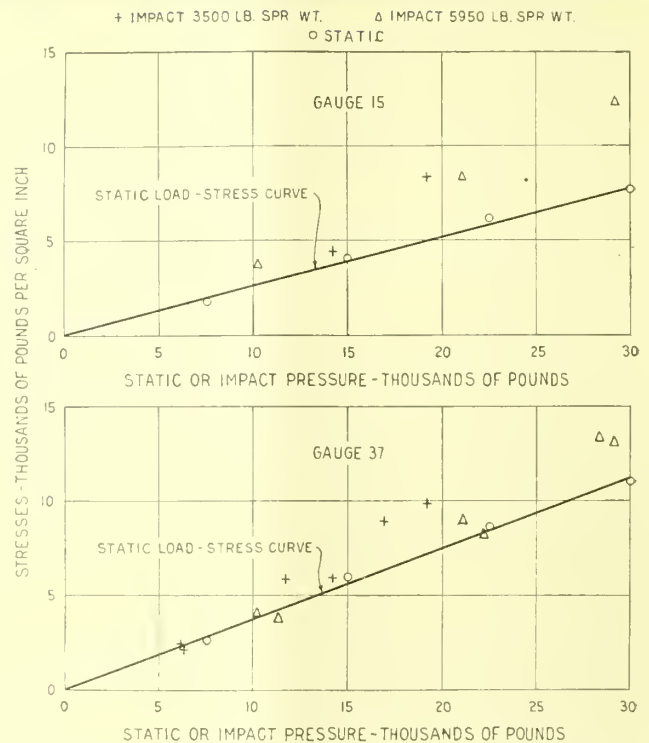


FIG. 46.—COMPARISON OF STRESSES IN REINFORCING TRUSSES OF FIRST SLAB FROM STATIC AND IMPACT LOADS. (SEE FIGS. 5, 11, AND TABLE 1)

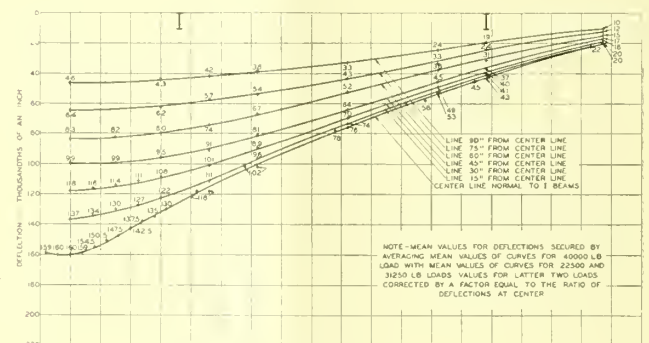


FIG. 47.—COMPOSITE OR TYPE DEFLECTION CURVES ON CENTER LINE NORMAL TO I BEAMS AND AT 15-INCH INTERVALS FOR STATIC LOAD OF 40,000 POUNDS

River Bridge Joint Commission who cooperated in the tests have suggested that analysis be made in the following manner:

By taking the deflections along the axes which are symmetrically located in the slab, composite or type curves may be developed as shown in Figure 47.

By developing graphically successive differentials of the original or type curves, by the tangent or other method, type curves for moment, shear and load distribution may be developed. An example of this for the first slab is shown in Figure 48.

As deflections were measured at shorter intervals along the axes of the second slab and as the deflections were measured by Ames dials fixed in place it is probable that better results would be obtained by using the deflection curves of the second slab. Composite curves for this slab are shown in Figure 47.

(Continued on p. 189)

FURTHER TESTS OF VIBROLITHIC CONCRETE

By the Division of Tests, United States Bureau of Public Roads. Reported by L. W. TELLER, Engineer of Tests, and C. E. PROUDLEY, Assistant Engineer of Tests

COMPARATIVE transverse bending tests on normal and Vibrolithic concrete have recently been completed by the Bureau of Public Roads. The tests were carried on at the Arlington Experimental Station of the bureau and the April, 1926, issue of PUBLIC ROADS contained a progress report, giving the data obtained as a result of tests of 28-day specimens. Since then the investigation has been concluded with the testing of a duplicate series of one-year specimens. This report is concerned largely with these later tests and includes data obtained in auxiliary investigations which have been made.



FIG. 1.—SLABS AND APPARATUS AT THE BEGINNING OF THE ONE-YEAR TESTS

EFFORT MADE TO ELIMINATE ALL VARIABLES EXCEPT THOSE BEING STUDIED

The object of the investigation was to obtain data on the resistance to rupture by bending, of specimens of different mixes fabricated by each of the two methods. These tests must not be considered, however, as a complete comparative study of Vibrolithic and normally finished pavements. Only one kind of aggregate was used and there is no evidence as to the effect of other kinds. The normal slabs were finished by hand, and the results obtained apply only to this method of finishing. The effect of machine finishing was not investigated.

In these experiments every effort was made to eliminate all variables except those which were being studied and to have all operations performed under as nearly similar working conditions as possible.

The methods of fabrication and of testing were given in detail in the previous report and will be outlined only briefly here. The test specimens were slabs 36 inches wide, 72 inches long, and 6 inches thick, made of carefully proportioned mixes. The coarse aggregate was limestone from Frederick, Md., graded from 2 inches to $\frac{1}{4}$ inch. The fine aggregate was clean and fairly coarse Potomac River sand. A carefully blended Portland cement of good quality was used. A correction was made for the bulking of the sand. The consistency for both types of slabs was maintained at about a 2-inch slump, or a flow of 110 to 115 on the 30-inch flow table, using a $\frac{1}{8}$ -inch drop.

Uniformity of the quality of the cement used throughout the tests was indicated at one year as at 28 days

by tests of 1:2 Ottawa sand mortar beams. These mortar tests are shown in Table 3 and Figure 2, and show that the cement was of satisfactory uniformity.

The concrete for all slabs was placed in the forms, tamped and struck off with a straight 2 by 12 inch strike board worked lengthwise over each group of five slabs. The normal finishing consisted of belting lengthwise over the slabs with an 8-inch rubber belt. The Vibrolithic concrete was covered uniformly with the Frederick limestone of 2-inch to 1-inch size at the following rate:

Mix	1:1½:3½	1:2:3½	1:2:4	1:2:4½
Pounds per square yard..	50	50	45	40

The special slatted platforms were placed on this stone and the vibrators were run over them, according to the patented process. The irregular mortar surface left on the removal of the platforms was smoothed down with a long-handled steel float, after which the surface was belted.

CURING OF SLABS AND TRANSVERSE BENDING TESTS

As soon as their hardness would permit, all slabs were covered with wet burlap, kept damp until the following day when they were covered with damp earth. This earth was kept damp for 28 days, after which it was removed and the slabs for testing at 28 days were broken.

The rest of the slabs, i. e., the one-year specimens, were left on the original subgrade during the remainder of the year and were covered with earth and kept wet for the 28 days immediately prior to testing in order that the concrete in all the specimens should be in as nearly a uniform condition of moisture as possible at the time of the test. Figure 1 shows the appearance of the general layout of the test sections at the beginning of the one-year tests.

The test data for the slabs, as given in Tables 1, 2, and 3, were obtained under practically the same conditions as the data which appear in Tables 1, 2, and 3 of the 28-day report.¹ The testing machine provided a span of 60 inches and applied the load at the third points over the entire width of the slab. The load was applied by means of a hydraulic jack and its magnitude was measured by the deflection of a pair of calibrated steel beams. The load causing failure was the sum of three components—the load applied by the jack, the dead load of the knife edges, jack and calibrated beams, concentrated at the points of application, and the uniform distributed load of the slab itself. A refinement in computation was introduced in the one-year tests, however, by using the true weight of each specimen in determining the dead load due to the slab instead of using an average figure for normal concrete and another average for Vibrolithic concrete, as was done previously in the 28-day tests. This refinement resulted in a very small difference in unit stress in most

¹ See Public Roads, vol. 7, No. 2, April, 1926.

TABLE 1.—Summary of data of the transverse bending tests on normal concrete slabs

Mix	Slab No.	Surface in tension	Depth	Section modulus I c	Total load at rupture	Modulus of rupture		Strength ratio $\frac{\text{Tension in top}}{\text{Tension in bottom}}$	Variation of individual tests from average of groups
						Individual slabs	Averages		
			<i>Inches</i>	<i>Inches³</i>	<i>Pounds</i>	<i>Pounds per sq. in.</i>	<i>Pounds per sq. in.</i>		<i>Per cent</i>
1:1½:3	6	Top	6.28	240.6	16,334	679	734	1.029	7.5
	9	do	6.09	230.2	16,904	734			0.0
	17	do	6.24	241.6	19,126	792			7.9
	18	do	6.31	246.5	17,876	725			1.2
	19	do	6.29	245.0	18,153	741			1.0
	7	Bottom	6.18	231.1	17,470	756	713		6.3
	8	do	6.21	233.7	15,356	657			7.9
	10	do	6.05	221.1	14,610	661			7.3
	16	do	6.22	253.1	19,448	792			11.1
	20	do	6.41	247.8	17,320	699			2.0
1:2:3	27	Top	6.29	245.0	15,302	625	596	.945	4.9
	29	do	6.40	253.7	15,468	610			2.3
	36	do	6.12	226.4	14,482	640			7.4
	38	do	6.25	242.7	12,672	522			12.4
	40	do	6.45	253.2	14,723	581			2.5
	26	Bottom	6.32	240.6	18,151	754	613		19.5
	28	do	6.29	239.6	15,740	657			4.1
	30	do	6.29	238.7	15,745	660			4.6
	37	do	6.18	231.1	13,010	563			10.8
	39	do	6.42	249.0	12,988	522			17.3
1:2:3½	47	Top	5.90	216.3	13,645	599	337	1.055	6.0
	49	do	6.05	227.3	14,253	627			1.6
	56	do	6.05	223.3	15,648	761			10.0
	58	do	6.32	247.6	15,862	641			0.5
	60	do	6.22	236.1	14,624	619			2.8
	46	Bottom	5.99	216.7	14,318	661	604		9.4
	48	do	6.00	218.2	13,089	600			7.0
	50	do	6.20	231.6	13,476	582			3.6
	57	do	6.19	232.2	14,513	625			3.5
	59	do	6.18	231.1	12,772	553			8.4
1:2:4	67	Top	5.99	223.0	13,202	592	598	1.010	1.0
	69	do	6.04	227.0	13,569	598			0.0
	76	do	6.01	220.3	14,224	646			8.0
	78	do	5.99	223.0	13,334	598			0.0
	80	do	6.01	220.3	12,247	556			7.0
	66	Bottom	6.06	221.4	14,518	656	592		10.8
	68	do	6.01	219.3	15,605	712			20.3
	70	do	6.12	225.8	11,855	525			11.2
	77	do	6.00	218.2	12,040	552			6.8
	79	do	5.98	217.6	11,224	516			12.8
Average		Top						4.3	
		Bottom						8.9	

TABLE 2.—Summary of data of the transverse bending tests on Vibrolithic concrete slabs

Mix	Slab No.	Surface in tension	Depth	Section modulus I c	Total load at rupture	Modulus of rupture		Strength ratio $\frac{\text{Tension in top}}{\text{Tension in bottom}}$	Variation of individual tests from average of groups	
						Individual slabs	Average			
			Inches	Inches ³	Pounds	Pounds per sq. in.	Pounds per sq. in.		Per cent	
1:1½:3½	2	Top	6.18	237.2	20,709	873	880	917	0.922	0.8
	4	do	6.28	244.6	19,706	806				8.4
	11	do	6.02	221.5	21,460	969				10.1
	13	do	6.19	237.6	20,689	871				1.0
	15	do	6.02	221.5	19,552	883				3
	1	Bottom	6.24	234.6	22,077	941	954			1.4
	3	do	6.26	237.0	22,852	964				1.0
	5	do	6.25	233.4	20,949	897				6.0
	12	do	6.00	218.2	22,800	1,045				9.5
	14	do	6.12	226.8	20,889	921				3.5
1:2:3½	22	Top	6.12	232.8	17,335	745	719		0.850	3.6
	24	do	6.54	264.5	17,987	680				5.4
	31	do	6.14	230.2	15,914	691				3.9
	33	do	6.18	237.2	19,241	811				6.6
	35	do	6.20	234.7	15,628	666				12.8
	21	Bottom	5.92	212.6	18,758	882	846			7.4
	23	do	6.22	234.0	21,369	913				4.3
	25	do	6.28	237.6	20,505	863				7.9
	32	do	6.20	232.6	17,297	744				2.0
	34	do	6.09	225.0	18,582	826				5.7
1:2:4	42	Top	5.98	222.0	15,970	719	732		0.957	1.8
	44	do	5.86	217.0	15,225	713				2.6
	51	do	6.28	240.6	18,115	753				2.9
	53	do	6.12	232.8	17,486	751				2.6
	55	do	6.12	228.7	16,504	722				1.4
	41	Bottom	6.18	230.2	17,101	743	765			2.9
	43	do	6.02	220.4	17,551	796				4.1
	45	do	5.91	211.1	15,986	757				1.0
	52	do	6.14	228.2	18,146	795				3.9
	54	do	6.22	234.0	17,176	734				4.1
1:2:4½	62	Top	6.11	231.7	13,927	601	614		0.840	2.1
	64	do	6.39	252.6	14,277	565				8.0
	71	do	6.12	228.7	13,532	592				3.6
	73	do	6.11	231.7	15,256	658				7.2
	75	do	6.12	228.7	14,944	653				6.4
	61	Bottom	6.20	231.6	18,783	811	731			10.9
	63	do	6.21	233.7	15,915	681				6.8
	65	do	6.29	230.9	16,932	730				0.1
	72	do	6.01	219.3	16,293	743				1.6
	74	do	6.11	226.4	15,658	691				5.5
Average		Top							4.6	
		Bottom							4.4	

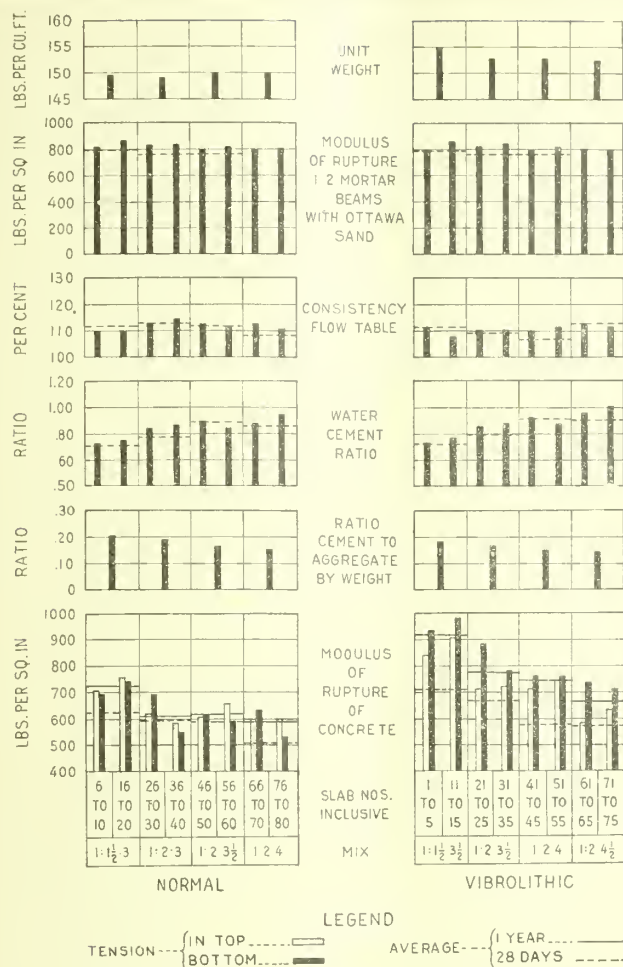


FIG 2.—RESULTS OF TESTS OF NORMAL AND VIBROLITHIC CONCRETE AND MORTAR BRIQUETTES AND BEAMS

cases. Figure 2 gives essentially the same data as shown in Table 3 and in addition, however, the averages of the 28-day test data are indicated by horizontal dotted lines.

MODULUS OF ELASTICITY DETERMINED FROM DATA SECURED IN BENDING TEST

At the same time that the one-year tests were made certain slabs were selected for determination of the maximum unit fiber deformation and modulus of elasticity. This was done by means of graphic strain gauges² set 0.25 inch from the surface of the slab between suitable gauge points. The gauges were placed at the top and bottom at each edge of the slab at the center of the span. Readings of these four gauges for progressive increments of load were the basis of computations of unit fiber deformations which were plotted with the computed unit fiber stresses. The initial modulus of elasticity was determined from these graphs. Figure 4 shows a typical graph of these data. Two slabs of each mix for both methods of finishing were tested in this manner, one being tested with the finished side in tension and the other with the subgrade side in tension. The values thus obtained are given in Table 4.

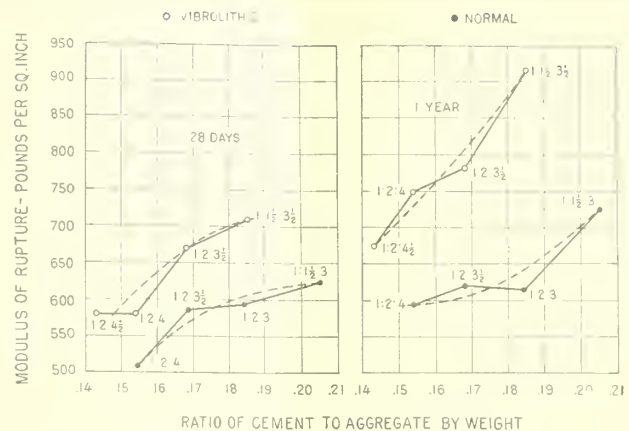


FIG. 3.—AVERAGE MODULUS OF RUPTURE COMPARED WITH CEMENT-AGGREGATE RATIO

TABLE 3.—Summary of all test data by groups on normal and Vibrolithic concrete

Finish.....	Normal concrete							
Mix.....	1:1 1/2:3		1:2:3		1:2:3 1/2		1:2:4	
Slab Nos., inclusive.....	6 to 10	16 to 20	26 to 30	36 to 40	46 to 50	56 to 60	66 to 70	76 to 80
Tension in top at rupture, pounds per square inch.....	707	753	618	581	613	654	595	600
Tension in bottom at rupture, pounds per square inch.....	691	746	690	543	614	589	631	534
Ratio of cement to aggregate by weight.....	0.205		0.184		0.168		0.154	
Water-cement ratio.....	0.73	0.75	0.85	0.87	0.90	0.85	0.88	0.95
Consistency, flow table.....	110	110	113	115	113	112	113	111
Modulus of rupture, pounds per square inch, 1:2 mortar using Ottawa sand.....	809	864	829	842	802	818	793	810
Weight per cubic foot.....	149.3		148.6		150.1		149.9	

Finish.....	Vibrolithic concrete							
Mix.....	1:1 1/2:3 1/2		1:2:3 1/2		1:2:4		1:2:4 1/2	
Slab Nos., inclusive.....	1 to 5	11 to 15	21 to 25	31 to 35	41 to 45	51 to 55	61 to 65	71 to 75
Tension in top at rupture, pounds per square inch.....	840	908	713	723	716	742	583	635
Tension in bottom at rupture, pounds per square inch.....	934	983	886	785	765	765	741	717
Ratio of cement to aggregate by weight.....	0.185		0.168		0.154		0.143	
Water-cement ratio.....	0.74	0.77	0.86	0.88	0.92	0.88	0.96	1.01
Consistency, flow table.....	112	108	111	111	110	112	113	112
Modulus of rupture, pounds per square inch 1:2 mortar using Ottawa sand.....	809	864	829	842	802	818	793	810
Weight per cubic foot.....	155.2		152.9		153.0		152.6	

COMPRESSION TESTS MADE ON FROZEN AND UNFROZEN CORES FROM SLABS

Cores were drilled from the fragments of 28-day specimens at about six months for compression tests and for alternate freezing and thawing. Results of these tests warranted the drilling of more cores from the one-year slabs and a repetition of the comparative compression and freezing tests. The cores were taken with a Calyx core drill and were 6 inches in diameter and approximately 6 inches high.

Shortly after the 28-day slab cores were drilled, an absorption test was made on each one. They were dried in an oven at about 80° C., cooled, weighed, immersed in water for 48 hours at 40° C. and again weighed. The results of these determinations are given in Table 5.

²GOLDBECK, A. T., POCKET STRAIN GAUGE GIVES STRESSES IN CONCRETE ROADS Engineering News-Record, Mar. 29, 1923

TABLE 4.—Modulus of elasticity determined by the transverse bending of slabs and by the compression of drilled cores¹

Normal				Vibrolithic					
Mix	Slabs		Cores		Mix	Slabs		Cores	
	No.	Modulus	No.	Modulus		No.	Modulus	No.	Modulus
		<i>Pounds per sq. in.</i>		<i>Pounds per sq. in.</i>			<i>Pounds per sq. in.</i>		<i>Pounds per sq. in.</i>
1:1½:3	6	3,550,000	6	3,200,000	1:1½:3½	5	5,040,000	5	5,980,000
	16	3,850,000	10	5,020,000		15	5,250,000	2	5,260,000
			18	4,500,000				11	6,944,000
Average		3,700,000		4,240,000	Average		5,140,000		6,061,000
1:2:3	26	4,310,000	28	4,640,000	1:2:3½	25	3,830,000	21	2,960,000
	36	3,320,000	29	4,720,000		35	4,340,000	31	4,500,000
			39	3,860,000				32	3,470,000
Average		3,810,000		4,410,000	Average		4,080,000		3,985,000
1:2:3½	46	3,100,000	50	5,280,000	1:2:4	45	4,420,000	42	4,400,000
	56	4,150,000	59	3,080,000		55	2,920,000	43	5,930,000
			60	3,380,000				55	4,100,000
Average		3,620,000		3,913,000	Average		3,670,000		4,810,000
1:2:4	66	4,680,000	69	4,720,000	1:2:4½	65	4,700,000	61	5,600,000
	76	2,970,000	70	6,000,000		75	3,630,000	62	4,300,000
			80	5,620,000				71	4,410,000
Average		3,820,000		5,447,000	Average		4,160,000		4,770,000

¹ Of the normal group slabs 16, 26, 46, and 66 were tested with the bottom surface in tension and slabs 6, 36, 56, and 76 were tested with the top surface in tension. Of the Vibrolithic group slabs 5, 25, 45, and 65 were tested with the bottom surface in tension and slabs 15, 35, 55, and 75 were tested with the top surface in tension.

TABLE 5.—Absorption tests on normal and Vibrolithic concrete a age of 7 months

Normal				Vibrolithic			
Mix	Slab No.	Absorption	Average absorption	Mix	Slab No.	Absorption	Average absorption
		Per cent	Per cent			Per cent	Per cent
1:1½:3	88	3.3	4.1	1:1½:3½	83	3.1	2.5
	89	3.5			84	3.6	
	99	2.8			91	1.8	
	100	3.2			93	1.1	
	118	4.4			111	2.4	
1:2:3	119	4.8	1.7	1:2:3½	122	2.4	2.3
	167	6.1			123	4.1	
	169	2.0			163	1.3	
	128	1.3			164	2.6	
	149	1.7			141	2.7	
1:2:3½	150	.9	2.2	1:2:4	142	3.2	3.1
	140	1.1			151	1.0	
	149	4.0			153	3.3	
	158	1.0			152	3.9	
	160	1.8			154	.9	
1:2:4	160	3.3	2.0	1:2:4½	155	1.0	1.6
	159	1.6			156	4.3	
	140	1.6			157	4.2	
	149	.9			158	2.4	
	150	3.9			159	1.4	
Average		2.4	2.5	Average		1.1	2.4

In the first series of freezing tests, three cores of each mix of both normal and Vibrolithic concrete were immersed in water in containers and surrounded by brine at approximately -12°C . When frozen solid the containers were removed from the brine and permitted to thaw. After 10 alternations of such freezing and thawing, all of the specimens showed disintegration so they were thawed out, capped and tested in compression.

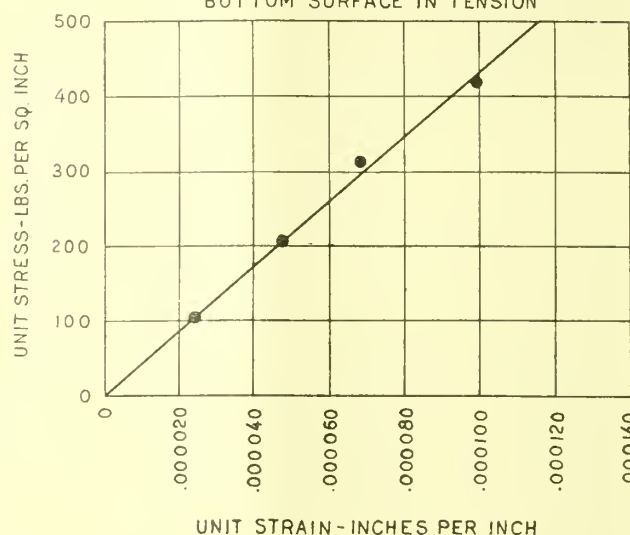
In the second series of freezing tests the specimens were kept saturated but not immersed during freezing. This change was made as an experiment, as it was thought that the rapid disintegration in the first test

SLAB NO. 26

1:2:3 NORMAL CONCRETE

 $E = 4,310,000$

BOTTOM SURFACE IN TENSION



SLAB NO. 75

1:2:4½ VIBROLITHIC CONCRETE

 $E = 3,630,000$

TOP SURFACE IN TENSION

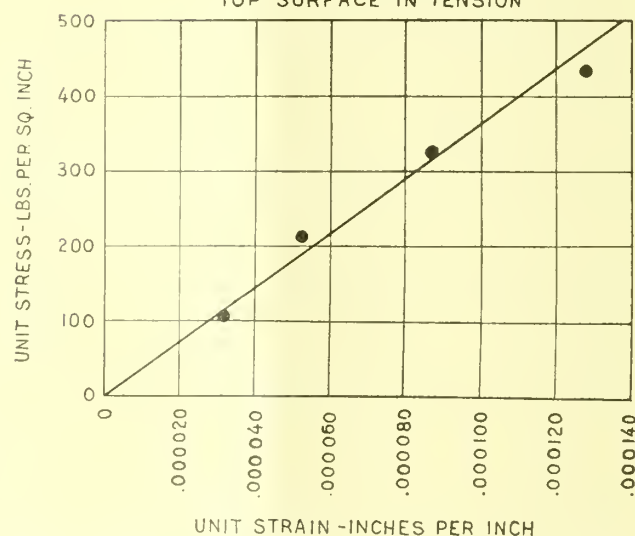


FIG. 4.—TYPICAL STRESS-STRAIN CURVES FROM THE TRANSVERSE BENDING TESTS

might be partially due to the expansion of the surrounding ice during freezing.

Unfortunately, due to leakage that occurred in some of the containers during the course of these later tests, brine entered some of the containers and came in contact with the lower end of some of the cores. It was impossible to determine with certainty which of the specimens had been exposed to the brine so that the effect of such exposure is not known. In view of this, none of the data obtained is above suspicion and is not included. However, even with this uncertainty admitted it can be said that it took approximately twice as long to produce disintegration by this method of freezing as by the first method.

The cores which were not frozen, of which there were five in the first series for each mix and kind of concrete,

were left in the field near the slabs from which they were drilled until they were about 12½ months old, at which time they were prepared for testing by immersing until a uniform condition of moisture was obtained. From the one-year slabs, three cores of each mix and kind were drilled, capped, and saturated with water and tested at the age of 17 months. Tables 6 and 7

TABLE 6.—Compression tests on 6 by 6 inch cores drilled from normal and Vibrolithic concrete at age of 13 months

Normal concrete cores				Vibrolithic concrete cores			
Mix	Slab No.	Load at failure	Average load at failure	Mix	Slab No.	Load at failure	Average load at failure
		Pounds per sq. in.	Pounds per sq. in.			Pounds per sq. in.	Pounds per sq. in.
1:1½:3	88	5,527	4,869	1:1½:3½	83	4,800	5,047
	89	5,340			84	5,580	
	99	4,985			84	4,882	
	99	3,707			91	4,580	
	100	4,788			93	5,391	
1:2:3	118	4,377	5,185	1:2:3½	111	5,142	5,322
	118	5,382			112	5,673	
	119	5,264			112	4,856	
	167	5,620			163	5,370	
	169	5,280			164	5,570	
1:2:3½	128	4,360	4,770	1:2:4	122	3,768	4,468
	129	4,378			123	5,381	
	138	5,710			123	4,640	
	140	5,823			131	4,561	
	140	5,578			133	3,988	
1:2:4	149	5,477	5,223	1:2:4½	141	4,400	4,900
	150	4,478			142	5,259	
	158	4,711			151	4,600	
	160	5,780			153	5,225	
	160	5,670			153	5,014	
Average			5,012	Average			4,934

¹ Total load over 200,000 pounds.

TABLE 7.—Compressive strength of 6 by 6 inch drilled cores after 10 alternations of freezing and thawing at age of 8 months

Normal concrete cores				Vibrolithic concrete cores			
Mix	Slab No.	Load at failure	Average load at failure	Mix	Slab No.	Load at failure	Average load at failure
		Pounds per sq. in.	Pounds per sq. in.			Pounds per sq. in.	Pounds per sq. in.
1:1½:3	88	2,730	2,653	1:1½:3½	83	2,025	2,047
	89	3,010			91	2,735	
	100	2,220			93	1,380	
	167	2,850			111	4,380	
	169	4,840			163	2,842	
1:2:3	119	2,590	3,427	1:2:3½	164	5,410	4,211
	128	4,020			122	2,475	
	129	3,680			131	4,275	
	138	2,985			133	1,396	
	149	2,190			141	2,770	
1:2:4	150	4,470	3,645	1:2:4½	142	3,970	4,027
	158	4,275			151	5,340	
Average			3,322	Average			3,250

TABLE 8.—Compression tests on 6 by 6 inch drilled cores from normal and Vibrolithic concrete at age of 17 months

Normal concrete cores				Vibrolithic concrete cores			
Mix	Slab No.	Load at failure	Average load at failure	Mix	Slab No.	Load at failure	Average load at failure
		Pounds per sq. in.	Pounds per sq. in.			Pounds per sq. in.	Pounds per sq. in.
1:1½:3	9	5,593	5,459	1:1½:3½	1	5,465	5,988
	10	4,993			2	5,496	
	18	5,790			11	7,004	
	28	5,849			21	5,728	
	29	5,770			31	4,809	
1:2:3	39	5,067	5,562	1:2:3½	32	5,320	5,286
	50	5,059			42	5,105	
	59	4,492			43	4,573	
	60	5,783			55	5,306	
	69	4,536			61	5,490	
1:2:4	70	3,262	4,311	1:2:4½	62	4,378	4,773
	80	5,136			71	4,451	
Average			5,111	Average			5,261

give the test results on the unfrozen and frozen specimens, respectively, while in Table 8 are shown the data from the compression tests at 17 months.

As a check on the modulus of elasticity of the concrete in the slabs as determined by the strain gauge measurements, tests to determine the modulus of elasticity in compression were made on each of the unfrozen cores from the one-year slabs prior to compression test. This was done by means of a mirror extensometer³ and the results are given in Table 4 where a comparison may be readily made.

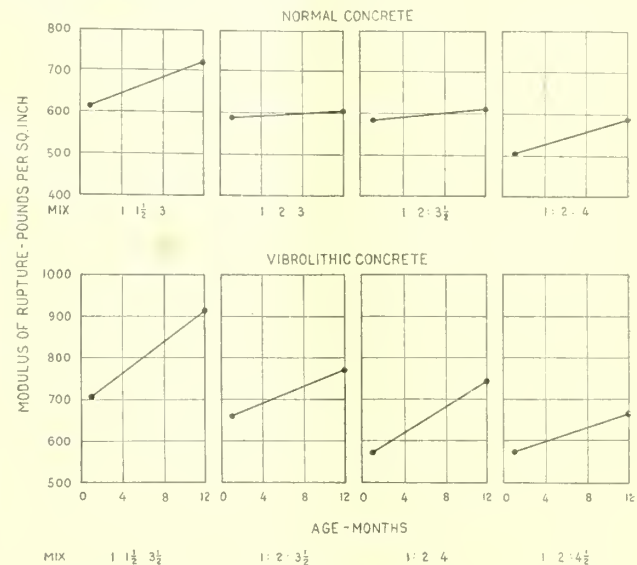


FIG. 5.—DIAGRAM SHOWING INCREASE IN MODULUS OF RUPTURE FROM 28 DAYS TO ONE YEAR FOR VARIOUS MIXES OF NORMAL AND VIBROLITHIC CONCRETE

It was thought that if the true section modulus of the slabs could be determined for those slabs which were honeycombed (for a discussion of the causes of honeycombing see the report on the 28-day tests) the computed strength would be more nearly the true average than when determined by the section modulus computed for a solid area. To study this effect half-size drawings of the broken section of the slabs in question were made by two observers working independently, and the areas of doubtful density carefully measured and located on the sketch. These are shown in Figure 6, the cross sections being shown in the position in which they were tested.

The area of these spaces and their distance from the neutral axis, the location of which was determined by a mechanical integrator for this part of the study, furnished data for a correction of the moment of inertia of the slab. Thirty-three of the most noticeably honeycombed specimens of both types of concrete were selected for investigation, and the results of the stress computations, although not published with this report, are discussed later.

TEST DATA DISCUSSION

A study of Figure 2 shows some interesting relationships between slab strength, water-cement ratio, consistency, and unit weight. As the water-cement ratio increases the slab strength decreases and the corresponding changes are in accord with the generally accepted theory in regard to this ratio. In the normal slabs, the richest mix with relatively high strength has

³ JOHNSON, A. N., DIRECT MEASUREMENT OF POISSON'S RATIO FOR CONCRETE. Proceedings A. S. T. M., vol. 24, Part II, p. 1024. 1924.

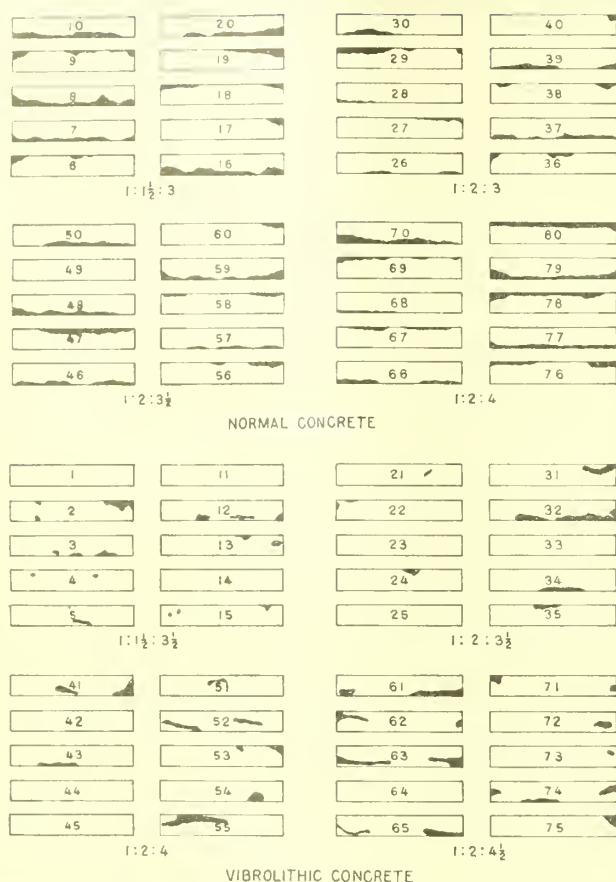


FIG. 6.—AREAS OF QUESTIONABLE DENSITY IN TEST SLABS, AS AVERAGED FROM THE SKETCHES OF TWO INDEPENDENT OBSERVERS. THE SLABS ARE SHOWN IN THE POSITION IN WHICH THEY WERE TESTED

a water-cement ratio of about 0.74. The remaining three mixes are comparatively close on strength and the water-cement ratios are 0.86, 0.87 and 0.91 which, from the standpoint of practical application are not greatly different. In the Vibrolithic specimens, the range in strength is greater as is also the range in water-cement ratio.

In comparing the consistencies it should be repeated that the concrete was mixed with the intention of having a flow of 110 to 115. This average was maintained with the exception of the 1:1½:3½ concrete used for slabs 11 to 15, inclusive. Here, in spite of a slightly higher water-cement ratio the slab strength is greater than the corresponding slabs 1 to 5. A similar condition is evident between the 1:2:3 and 1:2:3½ mixes of normal concrete. It may be that the measured consistency is more nearly an index of the actual water-cement ratio in the mixed concrete than can be obtained from a computation using the amount of water, moisture in the sand and volume of cement introduced into the mixer drum. High temperatures existing at the time these slabs were laid probably caused an unusually high rate of evaporation with consequent lower water-cement ratio and dryer consistency.

The unit weight determinations (Table 3) show that the addition of top stone in the Vibrolithic slabs resulted in a concrete of higher weight per cubic foot.

The quality of the cement as indicated by tests of the 1:2 mortar beams at one year was quite uniform, the maximum variation from average strength being

5.2 per cent and the average was 2.2 per cent. The test beams were 2 inches wide, 3 inches deep and 18 inches long, and were tested on a 15-inch span with center loading. Five specimens constituted a set.

STRENGTH OF VIBROLITHIC AND NORMAL CONCRETE SLABS COMPARED

The above discussion refers merely to the concordance of the slab strengths with other data relating to the mixtures. A comparison of the normal concrete with the Vibrolithic concrete may be made from Figure 2 or Table 3, but is more readily seen by referring to the dotted average curves in Figure 3. Considering first the strengths obtained for equal quantities of cement per unit volume of aggregate, the ordinates corresponding to the abscissa 0.154 representing a 1:2:4 normal concrete shows Vibrolithic concrete to be approximately 123 per cent of the normal concrete strength and in the richer mix at 0.184 the strength ratio is about 143 per cent. The corresponding ratios on the 28 day curves of Figure 3 are approximately 120 per cent and 117 per cent, respectively.

A more practical comparison is that showing the amount of cement required to give equal strength. Taking a modulus of rupture of 700 pounds per square inch for the comparison, the cement-aggregate ratio for Vibrolithic concrete is about 0.148 and for normal concrete is 0.201, an increase of about 36 per cent in cement requirement for the normal concrete. Or, expressed as a percentage of the normal requirement for concrete, the Vibrolithic concrete is equal in strength when 74 per cent of the cement required for normal concrete is used. For a modulus of rupture of 600 pounds per square inch at 28 days, the corresponding figures were 19 per cent and 84 per cent.

With respect to the relation between the tensile strength of the concrete in the top and in the bottom surfaces of the slabs of the two types, it will be noted that a change occurred between the 28-day and the 1-year tests. The earlier tests showed that in normal concrete the tensile strength in the bottom of the slab was only 87.7 per cent of that in the top and in Vibrolithic concrete it was 98 per cent. At one year, however, the resistance to tension in the bottom of the normal concrete specimens was 99 per cent of that in the top, whereas in the Vibrolithic concrete the resistance to tension in the bottom is 112 per cent of that in the top. In other words, although the resistance to tension in the bottom of the normal concrete slabs was considerably less than for the top of the slabs at 28 days, the results at one year show practically equal strength in top and bottom. The Vibrolithic concrete which showed practically equal resistance to tension in top and bottom at 28 days was considerably stronger in tension in the bottom fibers than in the top. A possible explanation of the more rapid growth in strength of the bottom of the slabs than occurred in the top for both normal and Vibrolithic is that the bottom being next to the subgrade had more water readily available for continuous hydration of the cement than the top which was exposed to the drying effect of the atmosphere for a period of 10 months.

EFFECT OF HONEYCOMBING DISCUSSED

The above comparisons are based on the averages of all specimens tested even though some varied considerably from the average. The majority of the specimens showing a considerable variation were above the average strength and, as will be brought out in later discussion,

it is probable that the strengths obtained are generally as accurate an indication of the strength of these slabs as those of apparently more consistent strength. These values, therefore, have been included in the average just as was done in the report of 28-day tests. Except for three or four of the normal concrete slabs which were tested with tension in the bottom, the percentage variation in both normal and Vibrolithic specimens averages about the same, indicating fairly satisfactory agreement in the slab tests as a whole. It also indicates that the product of either the Vibrolithic process or hand finishing is reasonably uniform in this investigation except for the few instances noted.

In the report of the earlier tests it was pointed out that those slabs whose cross sections were reduced by void spaces, or honeycombing in general, showed the lower modulus of rupture values. This same condition exists in the one-year specimens, as may be seen in the outline drawings of the sections in Figure 6. These outlines are the average result of independent sketches by two observers, and are not claimed to be exact, the solid portions merely representing areas of questionable density. They serve, however, to assist in explaining some of the apparently erratic slab strengths seen in Tables 1 and 2.

For example, slabs Nos. 8 and 10 exhibit relatively greater honeycombing in the bottom than do Nos. 7 or 20, and the strengths of Nos. 8 and 10 are lower than in the other two. Slab No. 16 showed the highest strength in this group tested with bottom in tension, and in fact, is equal to No. 17 which is apparently quite dense, yet inspection reveals the highest percentage of voids in any of the $1:1\frac{1}{2}:3$ normal concrete slabs. This inconsistency might be considered sufficient reason to exclude it from the average.

In the $1:2:3$ concrete slabs No. 39 is the only specimen which seems to be seriously honeycombed and it is seen has the lowest strength of this group. Slab No. 37 is sufficiently porous to be mentioned and is the next lowest in modulus of rupture. An inconsistency exists in this group also, slab No. 38 showing as low a strength as any but being apparently quite dense.

The next group, $1:2:3\frac{1}{2}$ normal concrete, has slabs Nos. 50, 48, 47, 46, and 59 noticeably honeycombed. No. 46, which shows the highest strength of those tests with the bottom in tension, exhibits a fair amount of void space at the bottom and might be considered somewhat inconsistent. The other specimens are fairly well in accord with respect to strength and apparent density.

The leanest of the normal concrete, which might be expected to be less dense than the rest, is considerably honeycombed in slabs Nos. 70, 77, 78, 79, and 80. Of the slabs mentioned, No. 79 is the lowest in strength, No. 70 is nearly as low, and No. 78 has a strength equal to the average of the group. Slabs Nos. 80 and 77 are below the average of the group. No. 76 which was fairly dense shows quite high strength, as does also No. 68 which has only a thin line of questionable area on its lower face.

In the Vibrolithic concrete specimens very little difference in density is noticeable until slab No. 32 of the $1:2:3\frac{1}{2}$ mix is reached. This slab is the lowest in strength of those tested with tension in the bottom. With the exception of this slab, it will be noticed in the $1:1\frac{1}{2}:3\frac{1}{2}$ and $1:2:3\frac{1}{2}$ mixes that the highest percentage variations from the average are due to strengths greater than the average.

The $1:2:4$ mix is not so uniform in density, especially Nos. 41 and 55. The effect, however, of this condition is not great and several inconsistencies could be pointed out but the maximum percentage variation in this group being only 4.1 per cent, the slabs are probably typical and the results reasonably correct. It is most readily apparent in this group that the Vibrolithic process tends to move the void spaces upward, which removes these less dense areas from the region of the maximum stress. This characteristic of Vibrolithic concrete was pointed out in the report of tests on 28-day specimens.

The next group of slabs is of $1:2:4\frac{1}{2}$ mix and is quite inconsistent with regard to the density-strength comparison. More specifically, No. 64 exhibits practically no voids but has low strength, and No. 61, which seems to be badly honeycombed, had the highest strength of the group. Nos. 63 and 74 have comparatively large areas of poor density and are also comparatively low in strength. Further comparisons of a less striking nature may be made which would be essentially the same as those given above.

MODULUS OF RUPTURE AT 28 DAYS AND ONE YEAR

The change in the modulus of rupture of the various groups of slabs from 28 days to one year, as shown in Figure 5, is also of interest. As an indication of the growth in strength of the concrete, the average of all one-year strength tests of normal concrete specimens was compared with the average of all of the 28-day normal specimens and found to be about 10 per cent greater. Similar comparison of the Vibrolithic concrete shows the one-year specimens to average about 23 per cent greater than the 28-day specimens. The increase in strength of the various mixes is also shown in Figure 3.

The water-cement ratio was slightly higher for the specimens for the 1-year tests than it was for those for the 28-day tests. The average difference was an increase of $0.85 - 0.81 = 0.04$ for the normal concrete and an increase of $0.88 - 0.83 = 0.05$ for the Vibrolithic concrete. This would tend to make the average increase from 28 days to 1 year slightly less for both normal and Vibrolithic concrete than would be expected had the same water-cement ratios obtained for both test groups.

Although the data on the determination of the value of the modulus of elasticity are somewhat meager, yet there are some interesting indications in them. In general, the values range from 3,000,000 to 6,000,000 pounds per square inch for each type of concrete and seem to bear no relation to either the compressive strength or the modulus of rupture. The average values for E , determined from the strain gauge readings on the flexed beams, are slightly lower than the average values determined by compression on drilled cores. This is true for both types of concrete. Also the average value of E for the normal concrete is somewhat lower than the average value for the Vibrolithic, whether the determination is made by bending or by compression tests.

RESULTS FROM ABSORPTION, FREEZING, AND COMPRESSION OF CORES DISCUSSED

The absorption of the concrete as shown in Table 5 furnishes but little material for definite comment when considered alone. Without going deeply into a discussion of effect of voids on absorption it would seem that other things being equal the richer concrete would have the lower percentage of voids. Also, it is reasonable to suppose that for concrete of identical materials,

quantities, and mixing, that which is rodded or vibrated the most will probably give the lower absorption. These two effects have probably offset each other and made the average absorption for the normal concrete and the Vibrolithic concrete practically the same.

In the first series of freezing tests, disintegration was so rapid that no difference in the behavior of the two types of concrete could be noticed. As previously remarked, it was thought that the high compressive stress set up in the specimen by the expansion of the surrounding block of ice caused sufficient differential in deformation of the mortar and stone, each having a different modulus of elasticity, to destroy the bond. The appearance of the frozen specimens indicated such action. Table 7 shows the strengths obtained when these frozen cores were tested in compression. The values are so erratic as to mean practically nothing. When these cores were frozen disintegration began at the corners and this left rounded irregular ends which, even when carefully capped, made the results of compression tests of doubtful value. It is thought that the effect of freezing on the strength of concrete will be indicated far more precisely by specimens which are suitable for transverse bending tests.

As already stated, the second series of freezing tests was unsuccessful so that the data which it was planned to obtain on the relative behavior of the two types of concrete under conditions of alternate freezing and thawing did not materialize.

Cores drilled from the 28-day test slabs were tested in compression at the age of 13 months, and these data are shown in Table 6. Considerable variation is to be noted in the individual test results; the different mixes do not show the strength variations which would be expected, and the difference in strength between the normal and Vibrolithic specimens so consistently indicated in the bending tests does not appear at all in these compression tests. The inference is that compression tests on these drilled cores are not a good measure of the strength of the concrete and indicate little except that, in general, the concrete is of good quality. This is also true of the cores tested at 17 months, the data for which appear in Table 8, except that the average strength of the different mixes appears to be somewhat more in accord with what might be expected.

CONCLUSIONS

When drawing conclusions from the data obtained in this investigation the following fact should be borne in mind. The normal slabs were too dry for hand finishing and the high percentage of honeycombing found in them is the result of this condition. Had the mix of these slabs been wetter a higher degree of compaction would probably have been obtained, but it is probable that the gain in strength which might have been expected as the result of higher density would have been offset by the reduction in strength due to the increase in water content. On the other hand, the dry mix was well designed for machine finishing, and if machine methods had been employed the normal slabs would probably have been more uniform and dense, and their strength correspondingly higher. The difference between the two types of concrete would then have been less. In spite of this, however, an examination of the strength values for the normal concrete for any given age and mix will show that this concrete was of at least average strength.

In summarizing the results of this investigation, it may be said that:

1. For a given cement content the slabs constructed by the Vibrolithic method showed greater strength than those of normal concrete, when measured by the transverse bending test.

2. The Vibrolithic concrete slabs were somewhat more uniform in strength than those of normal concrete in both the 28-day and 1-year tests, both types showing greater uniformity at 1-year than at 28 days.

3. The specimens of Vibrolithic concrete showed a greater percentage of increase in the modulus of rupture from 28 days to 1 year than did those of normal concrete.

4. Absorption tests on 6 by 6 inch drilled cores indicate that there is practically no difference in the absorptive properties of the two types of concrete.

5. Compression tests on 6 by 6 inch drilled cores tested at the ages of 13 and 17 months do not show the difference in strength between normal and Vibrolithic concrete that is indicated by the transverse bending tests.

6. The value of the modulus of elasticity ranges from 3,000,000 to 6,000,000 pounds per square inch for both types of concrete. The average value of E is somewhat higher for the Vibrolithic concrete than for the normal concrete by both the bending and compression test determinations.

APPLICATION OF THE RESULTS OF THE INVESTIGATION IN THE DESIGN OF PAVEMENTS

It has been shown in this report that there was a difference in the modulus of rupture or resistance to bending of concrete placed by the two methods. The magnitude of this difference for the various mixes may be ascertained by comparison of Tables 1 and 2, and the graphical presentation of the same data in Figure 3. Corresponding data will be found in the 28-day report.⁴

For the two comparable mixes the difference may be expressed as a ratio with the following result:

Mix	Bending strength ratio = $\frac{\text{Average modulus of rupture (Vibrolithic)}}{\text{Average modulus of rupture (normal)}}$	
	28-day tests	1-year tests
1 : 2 : 3½.....	669 = 1.14 586 579	782 = 1.26 620 748
1 : 2 : 4.....	507 = 1.14	595 = 1.26

It can not be said that the difference found in these tests will hold for concrete constructed of other aggregates. But for the moment, let it be assumed that we expect to use such aggregates as were used in the Arlington tests and that differences in strength such as were found will be obtained with the two methods of placing. How then, can the data developed by this investigation be used in the design of pavements? From the data at hand we may design pavements of equivalent bending strength, by either of the two methods, if we use—

- (a) the same mix but change the cross-section, or
- (b) the same cross-section but change the mix.

⁴ See Public Roads, vol. 7, No. 2, April, 1926, pp. 38 and 41.

Employing the first method, the knowledge of the modulus of rupture of concrete of a given mix finished by the two methods supplied by this investigation may be used to determine the required thickness of each type by means of the so-called corner formula.

The use of this simple formula for the design of road slabs is based upon the assumption that the most severe condition of loading likely to occur is that of a load concentrated at the extreme corner of a rectangular panel of the slab when, for one reason or another, the subgrade offers practically no support to the corner.

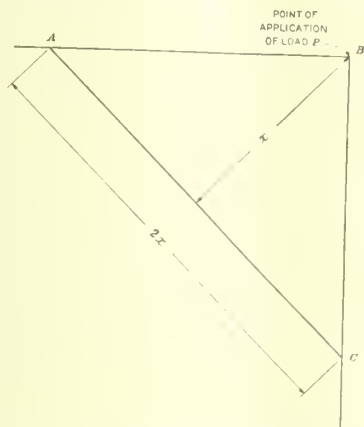


FIG. 7.—DIAGRAM FOR USE WITH DEMONSTRATION OF CORNER FORMULA

Under this condition the corner of the slab acts as a cantilever of uniform strength supporting a load, P , at the extreme corner. (Fig. 7.) At the distance, x , measured diagonally from the corner along the bisector of the right angle of the corner, the bending moment is then Px . This bending moment is resisted by the resisting moment of the section AC , which has a length of $2x$, and a depth of d , the depth of the slab.

The resisting moment is, therefore, equal to

$$\frac{2x \cdot d^3 S}{12 \cdot 2} = \frac{S x d^2}{3}$$

in which S is the unit tensile stress in the top of the slab.

Equating the resisting and bending moments,

$$Px = \frac{S x d^2}{3}, \text{ or } d = \sqrt{\frac{3}{S} P}$$

As tests of the fatigue of concrete have shown that the material will eventually fail under repeated applications of a load which produces a stress equal to approximately 55 per cent of the modulus of rupture. It is not safe to use a value of S in the corner formula in excess of 50 per cent of the modulus of rupture of the material.

Applying the corner formula to the design of slabs finished by the two methods described in this report, it will be observed that Tables 1 and 2 give values of

the average modulus of rupture at one year of the normal and Vibrolithic slabs, respectively, for mixes 1:2:3½ and 1:2:4. Similar values at 28 days were given in the earlier report.⁵

The values with the corresponding allowable stresses (one-half of the modulus of rupture) are shown for each mix in the following table.

28-DAY TESTS

Mix.....	1:2:3½		1:2:4	
Type.....	Normal	Vibrolithic	Normal	Vibrolithic
Modulus of rupture.....	Lbs. per sq. in. 586	Lbs. per sq. in. 669	Lbs. per sq. in. 507	Lbs. per sq. in. 579
Allowable stress.....	293	335	253	289

1-YEAR TESTS

Modulus of rupture.....	620	782	595	748
Allowable stress.....	310	391	298	374

Either the 1-year or the 28-day values may be used for substitution in the corner formula to design slabs of equivalent strength, using concrete of the same mix finished by the two methods, but as the pavement is put in service after three or four weeks, it is believed that the 28-day test values are a safer basis for comparison. Using the latter values the design of slabs of uniform depth is illustrated as follows:

SLABS OF SAME MIX AND VARIABLE DEPTH

Assuming a maximum wheel load, P , of 9,000 pounds and a 1:2:3½ mix, the depth of slab required is as follows:

For normal concrete—

$$d = \sqrt{\frac{3 \times 9,000}{293}} = 9.6 \text{ inches}$$

For Vibrolithic concrete—

$$d = \sqrt{\frac{3 \times 9,000}{335}} = 9.0 \text{ inches}$$

That is, a slab 9 inches thick of the stronger concrete will be equal in flexural strength to a slab of the weaker concrete 9.6 inches thick.

With a 1:2:4 mix and the same wheel load, the equivalent depths will be

For normal concrete,

$$d = \sqrt{\frac{3 \times 9,000}{253}} = 10.3 \text{ inches}$$

For Vibrolithic concrete,

$$d = \sqrt{\frac{3 \times 9,000}{289}} = 9.7 \text{ inches}$$

⁵ Public Roads, vol. 7, No. 2 Apr., 1926, p. 38.

These differences in thickness may be expressed in terms of the quantity of concrete required to lay a lineal yard of 18-foot pavement as follows:

For the 1:2:3½ mix—

	Cubic yards of concrete
Normal slab, 9.6 inches thick, will require.....	1. 60
Vibrolithic slab, 9.0 inches thick, will require.....	1. 50

Difference 10

As for the 1:2:4 mix—

	Cubic yards of concrete
Normal slab, 10.3 inches thick, will require.....	1. 72
Vibrolithic slab, 9.7 inches thick, will require.....	1. 62

Difference 10

Thus, with either mix, the reduction in depth permitted by the use of the stronger concrete will make possible a saving of about 0.10 cubic yard of concrete per lineal yard of 18-foot pavement.

THICKENED EDGE PAVEMENTS

By tests and also by theoretical analysis it has been found that a given wheel load will produce a greater stress in a concrete slab if applied at the edge or at a corner, as above, than if the same load is applied in the center of the slab; if the depth of the slab is the same at all points. Such a design is uneconomical because it does not fully employ the strength of the central portions of the slab; and in order that the central portion may carry a stress as great as the edges it has been found that the center depth should be approximately seven-tenths of the edge thickness.

On this basis the uniform-depth design, previously described would be modified as follows:

Assuming—

9,000-pound wheel load.

1:2:3½ mix, 28 days old.

Allowable stress in normal concrete= 293 pounds per square inch.

Allowable stress in Vibrolithic concrete=335 pounds per square inch.

Center thickness=.0.7 edge thickness.

Edge thickness by the corner formula:

9.6 inches for normal concrete.

9.0 inches for Vibrolithic concrete.

Center thickness:

6.7 inches for normal concrete.

6.3 inches for Vibrolithic concrete.

For an 18-foot pavement with the thickened edge disappearing 2 feet from the edge, the following volumes of concrete are required:

Per lineal yard of pavement:	Cubic yards
Normal.....	1. 17
Vibrolithic.....	1. 10
Difference.....	.07

SLABS OF SAME CROSS SECTION BUT DIFFERENT MIXES

Using the second method, equal strength in slabs of the two types may be obtained by employing different mixes, and typical data for such alternate designs are supplied directly by this investigation.

The following table gives the average modulus of rupture of the various mixes of the two types at 28 days and 1 year:

Mix	Average modulus of rupture (pounds per square inch)			
	28 days		1 year	
	Normal	Vibrolithic	Normal	Vibrolithic
1:1½:3.....	624		723	
1:2:3.....	595		613	
1:2:3½.....	586	669	620	782
1:2:4.....	507	579	595	748
1:1½:3½.....		708		917
1:2:4½.....		578		673

From these values it will be seen that the following mixes have approximately equal flexural strength:

28 days—1:2:3½ normal and 1:2:4½ Vibrolithic

1 year—1:1½:3 normal and 1:2:4 Vibrolithic

As the equivalent designs, in this case, are of the same thickness there is no difference in the amount of concrete used; but the proportions of cement, sand, and stone are different, and hence there is a difference in cost.

To estimate the difference in cost, the quantities of materials required to produce 1 cubic yard of mixed concrete of the several mixes must be determined and a very satisfactory formula for making this determination has been advanced by Stanton Walker.⁶ This formula is based on the assumption known to be practically correct—that the volume of concrete produced by the mixture of cement, and fine and coarse aggregate, and water in given proportions is equal to the combined solid volumes of each of the ingredients.

Thus, if the number of 1-cubic foot sacks of cement entering into a cubic yard of concrete of given mix be designated as N , and if—

x =the ratio of the volume of mixing water to the volume of cement (water-cement ratio);

W_f =the weight of fine aggregate, in pounds, used with one sack of cement;

W_c =the weight of coarse aggregate, in pounds, used with one sack of cement; and—

S_f and S_c =the apparent specific gravities of the fine and coarse aggregates, respectively.

And, finally, the volume of voids in a 1-cubic foot sack of cement being approximately 0.5 cubic foot, Mr. Walker's formula is expressed as follows:

$$0.5N + Nx + \frac{NW_f}{62.5 S_f} + \frac{NW_c}{62.5 S_c} = 27 \text{ cubic feet.}$$

Or—

$$N = \frac{27}{0.5 + x + \frac{W_f}{62.5 S_f} + \frac{W_c}{62.5 S_c}}$$

Using this formula the quantities of materials required to produce 1 cubic yard of mixed concrete of each of the mixes tested in this investigation are as follows:

⁶ WALKER, STANTON. ESTIMATING QUANTITIES OF MATERIALS FOR CONCRETE. Bull. No. 1, National Sand and Gravel Association, Washington, D. C.

Quantities of materials required for one cubic yard of mixed concrete

	Mix					
	1:1½:3	1:1½:3½	1:2:3	1:2:3½	1:2:4	1:2:4½
Cement, sacks.....	6.82	6.31	6.18	5.73	5.35	5.00
Sand, cubic yards...	.38	.35	.46	.42	.40	.37
Stone, cubic yards...	.76	.82	.69	.74	.79	.83

In calculating the above quantities the average water-cement ratio actually used in the test slabs was employed. These values were:

Mix	Av. $\frac{W}{c}$ (all tests)
1:1½:3-----	0.72
1:1½:3½-----	.74
1:2:3-----	.82
1:2:3½-----	.86
1:2:4-----	.90
1:2:4½-----	.95

Using the above quantities and assumed unit prices of the several materials (average current figures taken from a monthly tabulation by the Engineering News-Record), the cost per cubic yard of concrete of each mix may be calculated as follows:

Cost of materials per cubic yard of concrete

Material	Mix					
	1:1½:3	1:1½:3½	1:2:3	1:2:3½	1:2:4	1:2:4½
Cement, \$0.57 per sack.....	\$3.88	\$3.60	\$3.52	\$3.27	\$3.05	\$2.85
Sand, \$1.34 per cubic yard.....	.51	.47	.62	.56	.54	.50
Stone, \$1.83 per cubic yard.....	1.39	1.50	1.26	1.35	1.45	1.52
Total.....	5.78	5.57	5.40	5.18	5.04	4.87

Using these costs we may now make an approximate estimate of the comparative costs of the mixes of equivalent strength, above indicated.

Thus—

Mixes of equivalent strength:	Cost	Mixes of equivalent strength:	Cost
Normal, 1:2:3½---	\$5.18	Normal, 1:1½:3---	\$5.78
Vibrolithic, 1:2:4½-----	4.87	Vibrolithic, 1:2:4---	5.04
Difference-----	.31	Difference-----	.74

Reduced to a square yard basis for an 8-inch pavement this difference becomes \$0.069 and \$0.164 for the 28-day and 1-year strength values, respectively.

These figures do not include any fabrication costs but, except for the cost of finishing after the concrete is placed, such costs would be the same for the two mixes compared.

To determine exactly what the difference would be it would be necessary to have these costs of the finishing operation by each of the two methods.

Other material prices would also influence the comparison somewhat and the above calculations simply show the approximate cost relation and illustrate one method of making practical use of these test data.

The purpose of the foregoing discussion has been to point out two methods by which the highway engineer may make practical use of data such as are included in this report. The designs employed in the calculations are typical and the unit values are averages, so that the comparisons of the two types of concrete are valid with respect to the particular materials and methods of finishing employed in these tests. It would not be proper to assume, however, that the particular comparisons are valid with respect to the two types of concrete produced under other conditions.

(Continued from p. 178)

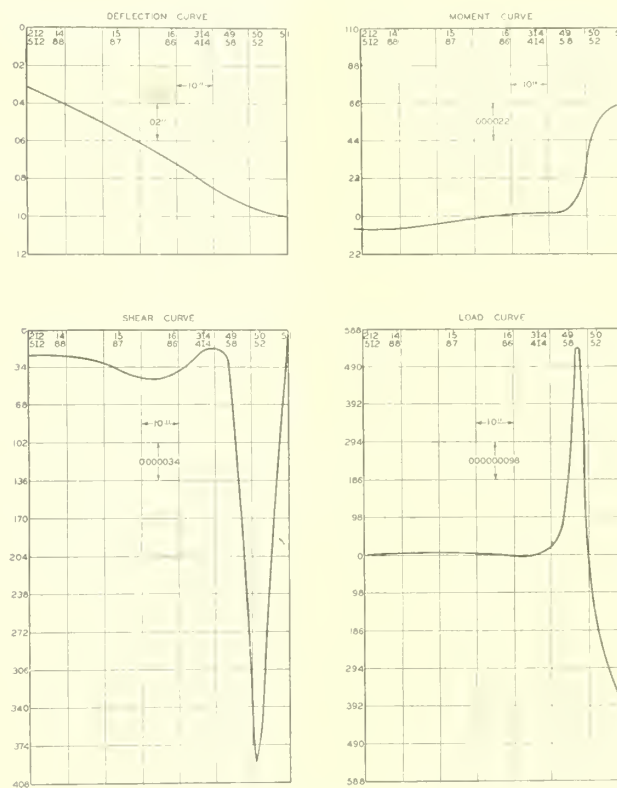


FIG. 48.—MOMENT, SHEAR AND LOAD CURVES DEVELOPED GRAPHICALLY FROM TYPE DEFLECTION CURVE ON AXIS NORMAL TO I BEAMS FOR 30,000-POUND LOAD. THE ORDINATES SHOWN ARE PROPORTIONAL TO MOMENTS IN FOOT-POUNDS, SHEAR IN POUNDS, ETC. THE ABSOLUTE VALUES ARE DEPENDENT UPON THE SCALES USED

By exercising proper care in the selection of the scales, successive curves may be developed from which, by giving due consideration to the variables for any individual slab, ordinates may be scaled which will give the moment, shear, and load at any point on the axes of the slab.

MOTOR VEHICLE REGISTRATIONS FOR THE FIRST SIX MONTHS OF 1927¹

State	Motor vehicles registered, individually and commercially owned, 1927			Other registered vehicles		Tax-exempt official cars as reported	Disposition of gross receipts (approximately allocated when details not reported)		Grand total motor vehicle registrations first 6 months of 1926	Increase of 1927 registration over 1926 (6 months) ⁴	
	Grand total motor cars, trucks, and road tractors	Passenger automobiles, taxis, and busses	Motor trucks and road tractors	Trailers	Motor cycles		Total gross receipts	Collection and administration			
										Number	Per cent
Alabama.....	211,385	184,059	27,326	1,061	287		\$2,863,482	\$100,819	\$1,112,060	13,783	7.0
Arizona.....	69,599	59,599	10,000	260	1,000		409,443	409,443	64,165	5,434	8.5
Arkansas.....	175,709	149,387	26,322	1,430	211	726	3,442,130	37,863	182,433	(1,526)	-0.9
California.....	1,584,723	1,385,894	198,829	29,570	8,122	9,293	7,862,541	1,738,841	3,061,850	125,153	8.6
Colorado.....	234,724	216,031	18,693	77	922		1,436,773	71,839	682,467	8,984	4.0
Connecticut.....	282,035	222,627	59,408	320	2,852	1,755	6,174,232	775,699	238,727	23,308	9.8
Delaware.....	42,784	34,705	8,079	204	1,069	3,387	5,510,733	589,647	1,235,018	2,481	6.1
Florida.....	373,482	315,009	58,473				3,476,424	89,859	3,386,565	(2,218)	-0.6
Georgia.....	260,079	226,146	33,933		730		3,476,424	89,859	3,386,565	288,618	9.0
Idaho.....	89,006	81,161	7,845	133	378	1,174	1,351,986	135,199	1,216,787	84,161	5.8
Illinois.....	1,366,060	1,192,286	173,774	3,063	5,318		14,301,788	9,471,998	1,217,265	148,795	12.2
Indiana.....	745,000	641,881	103,119	5,318	2,689	3,917	5,027,737	190,573	4,837,164	690,704	7.9
Iowa.....	660,888	610,415	50,473	130	1,526		9,379,051	598,663	8,780,388	12,006	1.9
Kansas.....	454,685	405,605	49,080	847	2,198	2,739	3,077,714	228,967	2,848,747	433,561	4.9
Kentucky.....	254,595	227,776	26,819	3,063	535	1,679	4,093,126	78,700	3,914,426	7,491	3.0
Louisiana.....	210,000	178,500	31,500		450		3,954,306	3,954,306	216,500	(6,500)	-3.0
Maine.....	141,605	119,261	22,344	755	886	1,117	2,248,145	101,944	2,146,201	13,139	10.2
Maryland.....	240,863	239,862	10,001	547	2,007	665	2,567,324	1,797,127	860,197	22,392	9.8
Massachusetts.....	697,404	611,747	85,657	409	5,305	800	10,870,798	1,150,000	9,720,798	69,608	11.1
Michigan.....	1,041,482	901,651	139,831	12,999	2,806	7,373	16,049,507	238,326	15,811,181	49,304	5.0
Minnesota.....	607,725	533,427	74,298	2,838	2,118		9,699,621	168,038	9,531,583	33,369	5.8
Mississippi.....	197,881	178,093	19,788	1,939	57	1,661	7,500,000	1,001,000	6,500,000	17,851	9.9
Missouri.....	609,849	547,479	62,370	1,939	1,466		10,870,798	1,150,000	9,720,798	583,450	4.5
Montana.....	91,701	78,396	13,305		118	248	1,020,380	32,647	987,733	(639)	-0.7
Nebraska.....	324,169	297,747	26,422	1,300	679	926	3,318,189	841,729	2,476,460	(5,500)	-1.7
Nevada.....	22,457	17,838	4,619	471	396		1,008,769	67,000	941,769	1,930	9.4
New Hampshire.....	86,618	70,200	16,418	477	1,059		1,008,769	67,000	941,769	7,639	9.7
New Jersey.....	639,339	524,550	114,789	1,398	5,504	5,400	11,382,556	308,256	11,074,300	575,237	11.1
New Mexico.....	49,315	43,241	6,074	109	172	619	495,100	48,707	446,393	3,985	8.6
New York.....	1,704,597	1,421,562	283,035	5,635	13,065	10,570	28,852,931	1,117,116	27,735,815	142,495	9.1
North Carolina.....	418,271	383,338	34,933	1,276	2,452		5,894,449	3,397,288	2,497,161	371,353	12.6
North Dakota.....	145,363	133,973	11,410		228		1,413,894	170,000	1,243,894	144,079	9.9
Ohio.....	1,459,815	1,299,606	160,209	10,466	8,284	5,600	8,784,294	674,783	8,109,511	89,059	6.5
Oklahoma.....	459,429	415,929	43,500		835		3,800,000		3,800,000	18,429	4.2
Oregon.....	204,835	189,756	15,079	(4)	1,598	2,068	5,922,214	125,000	5,797,214	135,641	4.7
Pennsylvania.....	1,425,424	1,241,247	184,177	3,319	10,526	3,175	22,925,010	614,655	22,310,355	1,326,052	7.4
Rhode Island.....	103,533	86,503	17,030	42	995	665	1,804,790	100,000	1,704,790	96,652	7.1
South Carolina.....	174,378	157,087	17,291	1,176	297	3,268	1,976,239	121,450	1,854,789	151,012	15.5
South Dakota.....	152,069	138,855	13,214		159	800	2,846,854	46,937	2,799,917	155,763	-2.4
Tennessee.....	265,842	243,443	22,399		643	3,238	3,451,293	41,544	3,409,749	38,067	16.7
Texas.....	930,110	833,881	96,229	6,980	2,335		14,236,439	674,783	13,561,656	46,060	5.1
Utah.....	84,450	72,927	11,523	149	432	650	603,771	132,000	471,771	81,580	3.2
Vermont.....	69,058	63,851	5,207	119	449		1,702,835	44,633	1,658,202	62,899	9.8
Virginia.....	299,924	256,338	43,586	255	1,023		4,849,833	67,141	4,782,692	277,125	8.2
Washington.....	348,628	298,639	49,989	1,451	1,914	4,434	5,676,974	88,172	5,588,802	326,500	6.8
West Virginia.....	205,121	182,816	22,305	292	1,043		2,235,585	139,699	2,095,886	183,788	11.6
Wisconsin.....	626,452	548,324	78,128		2,211	391	8,961,029	540,000	8,421,029	590,797	6.0
Wyoming.....	46,198	40,822	5,376		106	247	478,330		478,330	44,367	4.1
District of Columbia.....	91,873	80,183	11,690		814	1,915	344,931		344,931	89,857	2.2
Total.....	20,991,333	18,414,767	2,576,566	95,558	96,789	88,884	272,119,534	12,452,059	188,525,679	1,374,578	7.0

¹ Registration shown for 6 months in all States except North Carolina, which reports full year data, as registration year ends June 30.

² These first three columns record the regularly registered motor cars and trucks, excluding where possible the nonresident registrations and reregistrations of same vehicle.

³ Some States report decreases under previous year, indicated by parentheses and in the percentage column by a minus sign.

⁴ Interest and retirement payments on State road bonds except as indicated.

⁵ \$82,611 paid on State road bonds, remainder on county road bonds.

⁶ Includes undistributed funds.

⁷ Formerly reported as 407,777; the revised figure eliminates nonresident registrations included in 1926 data.

⁸ For Baltimore city streets.

⁹ Interest and retirement of county bonds.

¹⁰ For auto-theft bureau.

¹¹ Includes \$1,000,000 for bridge and tunnel commission.

¹² For auto-theft law enforcement.

¹³ Formerly reported as 490,000; the revised figure eliminates reregistrations included in 1926 published data.

¹⁴ Trailers included with trucks.

¹⁵ For highway motor patrol.

¹⁶ Includes refunds, \$23,392.

¹⁷ To U. S. Treasury, from which street appropriations are made for District of Columbia.

¹⁸ Revised total due to revisions in Florida and Oklahoma as noted.

ROAD PUBLICATIONS OF BUREAU OF PUBLIC ROADS

Applicants are urgently requested to ask only for those publications in which they are particularly interested. The Department can not undertake to supply complete sets nor to send free more than one copy of any publication to any one person. The editions of some of the publications are necessarily limited, and when the Department's free supply is exhausted and no funds are available for procuring additional copies, applicants are referred to the Superintendent of Documents, Government Printing Office, this city, who has them for sale at a nominal price, under the law of January 12, 1895. Those publications in this list, the Department supply of which is exhausted, can only be secured by purchase from the Superintendent of Documents, who is not authorized to furnish publications free

ANNUAL REPORTS

Report of the Chief of the Bureau of Public Roads, 1924.

Report of the Chief of the Bureau of Public Roads, 1925.

DEPARTMENT BULLETINS

No. 105D. Progress Report of Experiments in Dust Prevention and Road Preservation, 1913.

*136D. Highway Bonds. 20c.

220D. Road Models.

257D. Progress Report of Experiments in Dust Prevention and Road Preservation, 1914.

*314D. Methods for the Examination of Bituminous Road Materials. 10c.

*347D. Methods for the Determination of the Physical Properties of Road-Building Rock. 10c.

*370D. The Results of Physical Tests of Road-Building Rock. 15c.

386D. Public Road Mileage and Revenues in the Middle Atlantic States, 1914.

387D. Public Road Mileage and Revenues in the Southern States, 1914.

388D. Public Road Mileage and Revenues in the New England States, 1914.

390D. Public Road Mileage and Revenues in the United States, 1914. A Summary.

407D. Progress Reports of Experiments in Dust Prevention and Road Preservation, 1915.

*463D. Earth, sand-clay and gravel. 15c.

*532D. The Expansion and Contraction of Concrete and Concrete Roads. 10c.

*537D. The Results of Physical Tests of Road-Building Rock in 1916, Including all Compression Tests. 5c.

*583D. Reports on Experimental Convict Road Camp, Fulton County, Ga. 25c.

*660D. Highway Cost Keeping. 10c.

*670D. The Results of Physical Tests of Road-Building Rock in 1916 and 1917. 5c.

*691D. Typical Specifications for Bituminous Road Materials. 10c.

*724D. Drainage Methods and Foundations for County Roads. 20c.

*1077D. Portland Cement Concrete Roads. 15c.

*1132D. The Results of Physical Tests of Road-Building Rock from 1916 to 1921, Inclusive. 10c.

1259D. Standard Specifications for Steel Highway Bridges adopted by the American Association of State Highway Officials and approved by the Secretary of Agriculture for use in connection with Federal-aid road work.

1279D. Rural Highway Mileage, Income, and Expenditures, 1921 and 1922.

1486D. Highway Bridge Location.

DEPARTMENT CIRCULARS

No. 94C. TNT as a Blasting Explosive.

331C. Standard Specifications for Corrugated Metal Pipe Culverts.

MISCELLANEOUS CIRCULARS

No. 62M. Standards Governing Plans, Specifications, Contract Forms, and Estimates for Federal-Aid Highway Projects.

93M. Direct Production Costs of Broken Stone.

105M. Federal Legislation Providing for Federal Aid in Highway Construction and the Construction of National Forest Roads and Trails.

FARMERS' BULLETINS

No. *338F. Macadam Roads. 5c.

*505F. Benefits of Improved Roads. 5c.

SEPARATE REPRINTS FROM THE YEARBOOK

No. *739Y. Federal Aid to Highways, 1917. 5c.

*849Y. Roads. 5c.

914Y. Highways and Highway Transportation.

REPRINTS FROM THE JOURNAL OF AGRICULTURAL RESEARCH

Vol. 5, No. 17, D- 2. Effect of Controllable Variables upon the Penetration Test for Asphalts and Asphalt Cements.

Vol. 5, No. 19, D- 3. Relation Between Properties of Hardness and Toughness of Road-Building Rock.

Vol. 5, No. 24, D- 6. A New Penetration Needle for Use in Testing Bituminous Materials.

Vol. 6, No. 6, D- 8. Tests of Three Large-Sized Reinforced-Concrete Slabs Under Concentrated Loading.

Vol. 11, No. 10, D-15. Tests of a Large-Sized Reinforced-Concrete Slab Subjected to Eccentric Concentrated Loads.

* Department supply exhausted.

UNITED STATES DEPARTMENT OF AGRICULTURE
BUREAU OF PUBLIC ROADS

STATUS OF FEDERAL AID HIGHWAY CONSTRUCTION

AS OF

SEPTEMBER 30, 1927

STATES	FISCAL YEARS 1917-1927					FISCAL YEAR 1928					BALANCE OF FEDERAL AID FUND AVAILABLE FOR NEW PROJECTS				
	PROJECTS COMPLETED PRIOR TO JULY 1, 1927					* PROJECTS UNDER CONSTRUCTION					PROJECTS APPROVED FOR CONSTRUCTION				
	TOTAL COST	FEDERAL AID	MILES	TOTAL COST	FEDERAL AID	MILES	ESTIMATED COST	FEDERAL AID ALLOTTED	MILES	ESTIMATED COST	FEDERAL AID ALLOTTED	MILES	ESTIMATED COST	FEDERAL AID ALLOTTED	MILES
Alabama	\$ 20,061,371.68	\$ 9,615,099.94	1,400.2	\$ 84,878.41	\$ 40,723.98	0.2	\$ 7,048,755.05	\$ 3,368,591.51	409.8	\$ 39,289.25	\$ 19,544.62	0.2	\$ 2,862,514.95		
Arizona	11,809,950.70	6,447,169.27	800.8	267,759.35	192,916.61	14.1	1,448,605.72	1,086,686.44	67.8	34,528.88	17,534.44		2,584,294.24		
Arkansas	23,337,010.63	9,525,192.75	1,560.6	52,245.29	26,122.64	0.1	3,613,444.42	1,886,893.53	23.9	717,686.26	158,695.73	7.3	1,292,591.23		
California	35,128,269.04	16,967,056.82	1,305.3	826,569.60	429,604.25	29.8	7,526,051.48	3,463,353.06	150.1	31,568.35	18,941.01	0.1	3,677,316.86		
Colorado	15,497,121.91	7,934,298.91	829.0	91,935.74	48,486.64	0.9	6,387,143.17	3,121,842.16	280.3	35,740.10	20,057.34	2.9	2,577,645.95		
Connecticut	6,397,392.29	2,444,000.54	137.3	961,143.03	203,730.00	13.6	5,195,359.30	1,522,599.35	60.6	968,919.95	251,363.49	13.8	384,572.62		
Delaware	6,237,026.55	2,345,572.42	159.5	263,562.17	120,820.62	14.6	539,578.35	188,470.46	15.7	488,820.37	176,900.72	15.4	7,919.18		
Florida	7,476,856.31	3,627,912.60	245.1	1,517,818.43	734,206.79	32.7	8,217,445.95	3,507,757.54	176.2	777,845.42	269,730.00	18.0	844,788.07		
Georgia	31,951,436.50	15,101,232.40	2,173.6	2,340,425.97	1,113,809.87	86.1	7,407,445.95	3,668,582.92	296.5	379,747.45	188,028.25	0.1	339,508.07		
Idaho	13,225,515.45	7,075,527.16	835.5	149,006.57	87,998.40	24.2	2,808,373.89	1,743,751.70	187.9	666,191.56	397,052.37	36.3	150,479.77		
Illinois	48,538,982.16	22,781,516.60	1,530.8	706,567.81	344,072.82	53.8	14,556,820.36	7,016,546.35	505.6	2,527,747.47	1,258,103.41	86.7	1,930,460.64		
Indiana	23,372,717.74	11,238,568.20	732.5	409,186.56	191,269.03	13.3	16,868,915.82	8,075,101.26	507.8	1,136,510.75	373,205.37	43.1	252,993.14		
Iowa	34,305,138.86	14,395,603.75	2,484.4	709,235.61	344,072.82	53.8	15,357,626.43	6,680,555.95	549.3	539,012.31	98,386.66	15.2	11,942.82		
Kansas	37,442,061.61	19,730,829.48	1,495.2	671,046.37	299,030.45	36.7	14,839,934.09	5,871,973.53	762.0	1,056,933.04	381,057.44	99.1	250,052.10		
Kentucky	23,216,600.53	9,510,694.75	874.9	430,364.22	201,333.62	20.6	10,020,653.35	4,730,589.55	434.1	341,085.45	170,542.71	16.6	17,595.37		
Louisiana	15,877,552.20	7,093,892.21	1,178.7	931,554.63	442,316.21	37.9	3,421,391.18	1,523,675.45	95.6	1,208,085.09	604,042.54	13.1	621,789.58		
Maine	10,564,800.06	4,868,452.67	357.6	304,561.61	146,303.87	17.3	2,764,187.92	918,131.24	64.0	391,447.69	195,459.27	19.9	1,027,234.95		
Maryland	11,790,203.93	5,524,938.27	477.8				1,233,282.89	569,085.64	53.4	1,029,338.00	457,800.00	51.2	8,352.09		
Massachusetts	20,670,246.02	7,495,928.15	410.4	494,913.79	93,825.00	6.3	6,427,405.40	1,738,239.63	105.7	109,154.82	18,675.00	1.3	1,921,158.22		
Michigan	31,977,248.37	14,328,284.99	1,084.2	81,313.81	37,302.76	2.0	14,112,628.30	6,389,277.08	408.7	2,158,320.00	880,175.00	43.2	911,816.17		
Minnesota	45,099,648.47	19,046,145.67	3,543.5				7,533,620.17	2,158,686.90	392.0	443,394.96	6,000.00	13.5	501,476.43		
Mississippi	18,331,230.75	9,004,294.62	1,344.1	805,958.50	402,478.38	45.3	6,414,865.62	3,098,087.90	360.2	569,535.25	284,767.62	23.4	646,268.48		
Missouri	17,389,550.76	8,561,565.68	1,144.8	1,822,276.11	730,555.98	67.1	8,841,665.22	3,840,276.04	274.2	537,680.92	267,955.91	30.4	671,696.51		
Montana	12,854,995.72	7,487,128.69	1,151.6	206,459.10	152,862.59	25.3	3,202,940.78	2,277,574.70	243.5	1,837,105.41	1,059,313.87	153.2	4,189,324.15		
Nebraska	16,157,040.25	7,739,386.39	2,246.6	2,101,062.18	1,007,322.66	198.8	12,853,274.51	6,332,966.99	1,239.5	993,973.25	490,350.80	68.2	549,745.55		
Nevada	10,421,349.31	7,569,168.68	853.6	94,058.56	82,449.77	27.4	1,809,604.95	1,571,810.67	153.1	1,026,085.62	72,548.56	6.4	498,097.35		
New Hampshire	5,668,697.76	2,778,528.05	254.8	89,474.44	44,163.01	3.7	1,454,095.40	631,427.65	41.1	174,030.36	77,645.01	6.3	2,752.08		
New Jersey	22,228,240.08	7,495,354.48	316.3	1,359,957.39	277,170.00	18.5	6,962,629.39	1,601,398.56	104.7	94,656.00	28,095.00	1.9	1,631,103.86		
New Mexico	13,336,280.94	7,937,598.06	1,505.2	160,059.48	100,334.00	18.3	3,217,758.11	2,500,113.06	234.7	9,351,100.00	1,817,612.50	117.3	3,532,540.06		
New York	54,193,095.44	21,693,955.65	1,439.3	1,782,435.59	545,424.84	37.5	40,195,558.00	16,090,878.95	622.0	363,253.75	283.75	28.3	715,652.37		
North Carolina	35,295,849.21	14,518,903.16	1,460.1	747,052.66	349,898.15	28.6	3,144,127.66	1,482,854.57	80.4	1,777,080.50	353,253.75	210.6	135,579.76		
North Dakota	15,881,558.55	7,746,293.68	2,715.6	1,243,277.39	730,757.44	49.5	5,434,695.01	2,814,778.56	894.5	1,176,170.82	516,579.76	210.6	2,484,252.75		
Ohio	52,621,391.49	19,331,376.76	1,515.0	251,204.15	251,955.67	15.9	11,459,970.84	4,410,653.90	325.6	4,571,142.00	1,751,142.00	84.1	2,748,794.27		
Oklahoma	30,381,997.08	14,117,589.21	1,288.1	789,742.25	379,754.99	9.6	4,237,934.32	1,803,383.89	301.9	1,659,956.28	818,709.28	86.6	682,250.63		
Oregon	19,583,584.76	10,041,452.94	1,055.0	1,511,863.43	752,953.50	0.2	2,452,045.21	1,277,281.62	72.8	2,266,673.92	126,976.58	6.1	533,474.31		
Pennsylvania	77,725,174.22	36,317,650.32	1,534.3	869,406.01	282,528.10	22.9	18,738,648.66	5,695,885.02	370.7	2,924,606.80	951,386.84	43.8	1,427,152.72		
Rhode Island	5,203,033.38	1,998,479.05	116.0				1,815,941.88	400,533.74	31.1	47,199.34	128,638.67	7.6	435,512.53		
South Carolina	17,022,439.93	7,553,988.80	1,568.4	1,420,754.23	688,413.55	39.3	7,693,268.91	2,407,723.43	257.6	59,479.64	9,000.00	68.1	224,386.22		
South Dakota	19,282,063.24	9,507,525.54	2,502.9	109,739.28	57,825.39	29.1	4,443,470.27	2,420,862.36	747.1	494,552.37	272,003.80	68.1	128,636.31		
Tennessee	12,843,847.32	11,551,457.55	868.7	1,449,112.56	632,743.67	24.6	7,294,843.16	3,243,456.92	217.1	1,109,002.91	466,756.41	41.9	1,000,942.45		
Texas	78,190,246.37	31,566,960.48	4,466.4	3,543,794.64	1,555,087.85	124.6	14,425,669.14	6,515,476.58	497.3	2,928,218.13	1,216,404.43	100.2	4,249,773.71		
Utah	9,154,377.33	5,767,079.95	628.9	393,000.76	298,136.85	35.9	2,692,200.63	2,005,007.34	165.8	610,879.46	422,096.65	37.0	173,364.21		
Vermont	5,037,118.23	2,348,856.01	152.7	40,673.05	40,673.05	1.3	3,426,927.10	1,243,622.61	70.3	127,159.34	150,738.21	9.1	980.33		
Virginia	25,844,025.24	12,537,143.25	1,158.9	275,796.37	117,551.38	8.2	5,261,354.54	2,121,948.18	113.1	336,186.92	150,738.21	9.1	16,846.98		
Washington	18,184,505.97	8,246,551.95	711.1	847,151.85	415,736.32	5.3	2,918,690.33	1,330,600.00	66.5	1,243,406.64	508,000.00	48.3	776,413.73		
West Virginia	10,424,847.32	4,573,748.01	419.4				7,123,945.40	2,929,734.05	238.2	1,500,434.17	617,475.22	48.8	25,139.72		
Wisconsin	27,891,502.15	11,847,858.90	1,729.5	1,650,332.14	835,073.17	94.4	9,534,949.13	4,338,085.18	336.2	47,179.06	24,000.00	3.5	2,264,252.75		
Wyoming	12,650,712.15	7,139,257.05	1,315.9	218,889.07	143,801.91	24.2	2,740,927.37	1,753,760.77	130.4	197,169.20	126,582.59	39.6	337,230.73		
Hawaii	343,654.15	60,677,491.24	60,957.5	33,359,416.81	14,969,684.96	1,350.7	1,786,393.69	562,322.64	29.7	49,002,290.99	18,639,399.43	1,808.7	805,971.35		
TOTALS	1,154,740,601.48	510,007,601.48	60,957.5	33,359,416.81	14,969,684.96	1,350.7	358,889,558.44	150,019,212.47	14,135.3	49,002,290.99	18,639,399.43	1,808.7	50,954,011.90		

* Includes projects reported completed (final vouchers not yet paid) totaling: Estimated cost \$ 101,879,159.50 Federal aid \$ 44,022,355.53 Miles 4,214.9

PUBLIC ROADS

A JOURNAL OF HIGHWAY RESEARCH



UNITED STATES DEPARTMENT OF AGRICULTURE
BUREAU OF PUBLIC ROADS



VOL. 8, NO. 9



NOVEMBER, 1927



MIXING TOPSOIL WITH BITUMINOUS MATERIAL IN SOUTH CAROLINA EXPERIMENTS

PUBLIC ROADS

A JOURNAL OF HIGHWAY RESEARCH

U. S. DEPARTMENT OF AGRICULTURE

BUREAU OF PUBLIC ROADS

CERTIFICATE: By direction of the Secretary of Agriculture, the matter contained herein is published as administrative information and is required for the proper transaction of the public business

The reports of research published in this magazine are necessarily qualified by the conditions of the tests from which the data are obtained. Whenever it is deemed possible to do so, generalizations are drawn from the results of the tests; and, unless this is done, the conclusions formulated must be considered as specifically pertinent only to the described conditions

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R. E. ROYALL, Editor

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SURFACE TREATMENT OF TOPSOIL ROADS

REPORT ON COOPERATIVE EXPERIMENTS BY THE UNITED STATES BUREAU OF PUBLIC ROADS AND THE STATE HIGHWAY DEPARTMENT OF SOUTH CAROLINA

Reported by J. T. PAULS, Associate Highway Engineer

COOPERATIVE EXPERIMENTS UNDERTAKEN

LOCAL considerations of traffic and of economic necessity make it certain that relatively low-cost road types, constructed of local materials, will long continue to constitute a large proportion of our highway system. In some sections sand-clay and sand-clay-gravel surfaces predominate. Where conditions are favorable to these types they are entirely satisfactory, but there is a considerable mileage which is kept in a satisfactory condition only by constant and painstaking maintenance at considerable cost. Such road surfaces are extremely dusty when dry and frequently repair and reconstruction costs amount to large totals.

Topsoil roads have been built extensively in the South, where, in many sections, suitable surfacing material occurs as the thin upper layer of the soil, particularly on hills and knolls. The material contains a preponderance of small gravel particles from which a considerable portion of the original clay content has been removed by natural processes assisted by cultivation.

Naturally, all native deposits are not equally suitable for road building. The characteristics and amount of the clay as well as the gradation of the coarser particles influence the behavior of the material as a road surface. Highly plastic clay is not desirable in as large a quantity as where it is of a low degree of plasticity. An excessive amount of clay, while bonding well in dry seasons, is likely to cause muddiness and rutting during continued wet weather. A deficiency of clay, if not too marked, may prove satisfactory at the time of rains, but is apt to result in raveling when moisture is lacking. Methods for the determination of the quality of clays are not yet fully developed, but tests are available by means of which their properties may be estimated and compared.

Considerable research is being conducted to develop methods of eliminating dust and materially reduce maintenance costs on sand-clay or topsoil road surfaces. Experiments generally involve the use of bituminous materials for the purpose of reinforcing the existing surface either by impregnation or by the addition of a thin surface layer of greater resistance to the destructive agencies of nature and traffic.

SOUTH CAROLINA STATE HIGHWAY DEPARTMENT EXPERIMENTS¹

In 1924 the State highway department of South Carolina applied a surface treatment to a 7-mile section of the topsoil road between Spartanburg and Inman. A priming coat of light refined tar was first applied at the rate of one-fourth gallon per square yard followed by one-third gallon per square yard of either tar or asphalt applied hot. Crushed stone, one-fourth to 1¼ inches in size, was spread over the surface at a rate of about 50 pounds per square yard and the operation was completed by rolling.

The State work yielded such promising results that further experiments of this general nature were undertaken during 1925 by the South Carolina State highway department in cooperation with the United States Bureau of Public Roads.

A 7-mile section of Route 15 in Anderson County, beginning west of the city of Anderson, at the end of the bituminous pavement and extending west approximately 7 miles, was selected for a service test of six experimental sections. Details of these sections are shown in Table 1. Five other experimental sections were located in Spartanburg County on Route 19, beginning at the south city limits of Inman and extending north on the Spartanburg-Tryon highway to the State line. These five sections aggregated approximately 8 miles in length and were designed as shown in Table 2.



A DOUBLE SURFACE TREATMENT AFTER TWO YEARS SERVICE

Several methods of construction were adopted and a number of very different bituminous materials specified in order that the maximum amount of information might be derived with respect to their behavior when applied to road surfaces varying in composition and texture. In selecting mineral cover material, it was decided to use natural topsoil in comparison with local river sand on some of the sections.

PROPERTIES OF TOPSOIL STUDIED

Before the construction of these experiments was started, a survey was made of the condition of the topsoil surface of the roads selected. Samples of the material were taken from the surface and subgrade at intervals. The results of laboratory tests on these samples are given in Table 3.

A wide variation in the condition of the different sections was noted in the survey. The surface was smooth and well bonded on a major portion of each, but in some places it was loose and in others rough and potholed.

On experimental section 6, from station 280+00 to 320+00, scaling and crumbling was prevalent. The material in this location contained a large amount of

¹ See Surface Treatment of Roads—Hot Application Method, by N. S. Anderson, maintenance engineer, South Carolina Highway Department, Proceedings of the Fifth Annual Asphalt Paving Conference.

TABLE 1.—Details of construction and re-treatments of experimental sections on Route 15, all sections 20 feet in width

ORIGINAL TREATMENTS IN MAY AND JUNE, 1925

Section and station	Type and depth	Bituminous material		Covering
		Character	Gallons per square yard	
Sec. 1. 0+00 to 15+84	Mixed, 2 inches	Tar, 8 to 13 specific viscosity at 40° C	1.5	None.
Sec. 2. 15+84 to 34+32	do	Medium quick-drying oil	1.5	None.
Sec. 3. 34+32 to 108+24	Surface-treated	Tar, 8 to 13 specific viscosity at 40° C	.38	Topsoil, 34+32 to 85+00; sand, 85+00 to 108+24.
Sec. 4. 108+24 to 161+04	do	Tar, 18 to 25 specific viscosity at 40° C	.38	Topsoil, 134+00 to 161+04; sand, 108+24 to 134+00.
Sec. 5. 161+04 to 279+84	do	Slow-drying oil	.38	Sand, 161+04 to 255+00; topsoil, 255+00 to 279+84.
Sec. 6. 279+84 to 361+00	Double surface-treated	Primer: Tar, 8 to 13 specific viscosity at 40° C Second coat: Hot asphalt, 150 to 200 pen (Station 359+00 to 361+00, no primer)	.25 .33	Stone, 1/4 to 1 inch.

RE-TREATMENTS

Section and station	Date	Type and depth	Bituminous material		Covering
			Character	Gallons per square yard	
Sec. 1. 0+00 to 15+84	July, 1925	Surface-treated	Tar, 18 to 25 specific viscosity at 40° C	0.2	Sand.
Do	March, 1927	Mixed, 3 1/2 inches	do	1.1	
Do	do	Surface-treated	do	.2	Do.
Sec. 2. 15+84 to 34+32	November, 1925	Mixed, 2 inches	Slow-drying oil	1.1	
Do	March, 1927	Mixed, 3 1/2 inches	do	1.1	
Do	do	Surface-treated	do	.2	Do.
Sec. 3. 34+32 to 81+00	July, 1925	Scarified			
Do	November, 1925	Surface-treated	Tar, 18 to 25 specific viscosity at 40° C	.35	Do.
81+50 to 108+24	do	Surface-treated	do	.18	Do.
34+32 to 47+00	March, 1927	Scarified and surface-treated	Tar, 13 to 18 specific viscosity at 40° C	.40	Do.
80+00 to 94+00					
47+00 to 80+00	do	Surface-treated	do	.25	Do.
94+50 to 108+24					
Sec. 4. 134+00 to 161+04	July, 1925	Scarified			
Do	November, 1925	Surface-treated	Tar, 18 to 25 specific viscosity at 40° C	.37	Do.
108+24 to 134+00	do	do	do	.24	Do.
159+00 to 161+04	March, 1927	Scarified and surface-treated	Tar, 13 to 18 specific viscosity at 40° C	.40	Do.
108+24 to 159+00	do	Surface-treated	do	.25	Do.
Sec. 5. 161+04 to 279+84	November, 1925	do	Slow-drying oil	.2	Do.
Do	March, 1927	do	do	.2	Do.
Sec. 6. 279+84 to 353+00	November, 1925	do	do	.2	Do.

mica, which seemed not only to weaken the bond but also, by collecting in layers and pockets, to cause scaling of the surface. The South Carolina highway department has found that those portions of the surface containing high percentages of mica are slippery, unstable, and difficult to maintain during the wet season.

The portions of the surface showing the greatest mica content were located in the deeper cuts and on fills built with the soil from these cuts. In these locations the mica-bearing subsoil had been incorporated with the topsoil by the mixing during construction and maintenance.

The topsoil surface material on the experimental sections in Spartanburg County contained in general much less mica than the surface on portions of the Anderson County road. However, in the former county the surface of experimental section 11 between stations 729+00 and 791+25 scaled and crumbled badly during the progress of the treatment. This may have been the result of spreading a coat of finely crushed granite during the spring which at the time was insufficiently mixed with the clay binder underneath.

Table 3 shows that the clay content in the topsoil material ranged from a minimum of 8 per cent to a maximum of 32 per cent, the average being about 20 per cent. The surface from which the sample containing 8 per cent clay was taken was loose and badly broken. In the area represented by the sample having

32 per cent clay, the surface remained smooth and well bonded during dry weather but rutted somewhat during wet weather. It appears, by comparing the clay content with the condition of the surface, that less than 20 per cent clay in this topsoil did not in general give a stable and well-bonded surface during all seasons of the year. Clay was considered as including all particles of less than about 0.02 millimeter diameter and on the average constituted approximately twice as much material as would have been the case had the limit been taken as 0.005 millimeter diameter. A diameter of 0.005 millimeter is usually considered the upper limit under present soil terminology. The low dye-adsorption numbers obtained may be taken to indicate a clay low in stickiness and plasticity. This probably explains why such a large percentage of clay was needed to get a well-bonded surface.

The low moisture equivalent obtained indicates in general that the material dries out very quickly after a rain. This characteristic property of the topsoil surface was noted during construction.

Other physical tests add but little to this information, probably due to some extent to the fact that the proportion of sand is so large as to render inapplicable tests designed primarily for soils. In general, however, it is seen that for samples from poorly bonded locations the dye adsorption is 5 or less, the moisture equivalent is less than 8, and the lower liquid limit is less than 19.

LOCAL CONDITIONS CONSIDERED IN SELECTING BITUMINOUS MATERIALS

In selecting bituminous materials for the various sections, certain qualities were specified which were thought to be desirable with the methods of construction and maintenance adopted. For sections 1 and 2, materials were desired which would mix readily with the topsoil, permit working and smoothing with road machinery soon after being applied but which would harden with sufficient rapidity to produce a stable surface under traffic.

A thin mat surface, susceptible to further strengthening by subsequent treatments, was desired on sections 3 and 4. For this purpose tars were specified of such nature and consistency that they would penetrate the topsoil, thus minimizing the danger of displacement under traffic. Tars of two consistencies were used to determine their relative efficiencies.

The object on section 5 was the development of a surface somewhat similar to those of sections 3 and 4 except that a rather light grade of slow-curing oil was employed. With such a material it was anticipated that a rather plastic mat would result which could be shaped by dragging and thickened by later treatments.

On the double surface treatments, experiments 6 to 11, inclusive, the purpose was to form a stable mat of stone and bituminous material, firmly bonded with the topsoil surface. A rather fluid product, designed to penetrate the road and to dry with sufficient rapidity to permit early application of the second coat, was used as a primer. Over this a hot refined tar or asphalt

was sprayed, the primary function of which was to bind the relatively heavy layer of coarse aggregate.

Table 4 gives the results of tests made on the mineral cover materials and Tables 5 and 6 give the characteristics of the bituminous materials used on the several sections.



SCARIFYING TOPSOIL SURFACE FOR THE MIXING TREATMENT
CONSTRUCTION METHODS DESCRIBED

Practically the same methods were employed in the construction of experimental sections 1 and 2. The topsoil road surface in each case was scarified to a depth of 2 to 3 inches, after which this loosened material was pulverized by disk harrowing. About one-half of the loose material was then bladed to the sides of the road

TABLE 2.—Details of construction and re-treatments of experimental sections on Route 19; from station 0+00 to 33+00 the surface was 25 feet wide and on the remainder 19 feet wide

ORIGINAL TREATMENTS, JUNE, 1925

Section and station	Type	Bituminous material		Covering
		Character	Gallons per square yard	
Sec. 7. 0+00 to 82+50.....	Double surface-treated.....	{Primer: Tar, 8 to 13, specific viscosity at 40° C..... Second coat: Hot tar.....	$\frac{1}{2}$ $\frac{1}{2}$	$\frac{1}{4}$ to 1 inch stone.
82+50 to 436+00 ¹	Double surface-treated.....	{Primer: Cut-back oil..... Second coat: Hot asphalt, 150 to 200 pen.....	$\frac{1}{2}$ $\frac{1}{2}$	
Sec. 8. 436+00 to 512+00.....	Double surface-treated.....	{Primer: Cut-back oil..... Second coat: Hot tar.....	$\frac{1}{2}$ $\frac{1}{2}$	Do.
Sec. 9. 512+00 to 536+00.....	do.....	{Primer: Cut-back oil..... Second coat: Hot tar.....	$\frac{1}{2}$ $\frac{1}{2}$	
Sec. 10. 536+00 to 622+50.....	do.....	{Primer: Tar, 8 to 13, specific viscosity at 40° C..... Second coat: Hot tar.....	$\frac{1}{2}$ $\frac{1}{2}$	512 to 517, $\frac{1}{4}$ to 1 inch stone, 517 to 533, $\frac{1}{4}$ -inch chats, 533 to 536, $\frac{1}{4}$ to 1 inch slag.
Sec. 11. 622+50 to 791+25.....	do.....	{Primer: Tar, 8 to 13, specific viscosity at 40° C..... Second coat: Hot asphalt, 150 to 200 pen.....	$\frac{1}{2}$ $\frac{1}{2}$	
				536 to 553, $\frac{1}{4}$ to 1 inch slag. 553 to 622+50, $\frac{1}{4}$ to 1 inch stone. $\frac{1}{4}$ to 1 inch stone.

¹ This section was double surface-treated by the State highway department in 1925.

RE-TREATMENTS

Section and station	Date	Type	Bituminous material		Covering
			Character	Gallons per square yard	
Sec. 7. 38+00 to 82+50.....	November, 1925.....	Surface-treated.....	Tar, 18 to 25, specific viscosity at 40° C.....	$\frac{1}{2}$	Sand.
0+00 to 38+00.....	March, 1926.....	do.....	Tar, 8 to 13, specific viscosity at 40° C.....	$\frac{1}{2}$	Do.
82+50 to 436+00 ¹	March, 1926.....	Surface-treated.....	Medium quick-drying oil.....	$\frac{1}{2}$	Do.
Sec. 8. 436+00 to 512+00.....	March, 1926.....	do.....	Tar, 8 to 13, specific viscosity at 40° C.....	$\frac{1}{2}$	Do.
Sec. 9. 512+00 to 536+00.....	do.....	do.....	Tar, 13 to 18, specific viscosity at 40° C.....	$\frac{1}{2}$	Do.
Sec. 10. 574+50 to 622+50.....	November, 1925.....	do.....	Tar, 8 to 13, specific viscosity at 40° C.....	$\frac{1}{2}$	Sand, 553 to 574+50; 536 to 553, stone.
536 to 574+50.....	March, 1926.....	do.....	Slow-drying oil.....	$\frac{1}{2}$	Sand, 622+50 to 720, 735 to 750, 780 to 791+25; remainder $\frac{1}{4}$ - $\frac{3}{4}$ inch stone.
Sec. 11. 622+50 to 791+25.....	November, 1925.....	do.....			

¹ This section was double surface-treated by the State highway department in 1925.



PULVERIZING THE TOPSOIL AFTER SCARIFYING



FINAL APPLICATION OF BITUMINOUS MATERIAL PREPARATORY TO MIXING



FIRST APPLICATION OF BITUMINOUS MATERIAL TO A PORTION OF THE SOIL. SECOND APPLICATION TO FOLLOW AFTER SPREADING THE SOIL SHOWN IN WINDROWS



SHAPING AND COMPACTING

TABLE 3.—Analysis and tests of samples of topsoil surfacing from experimental sections

Soil No.	Section No.	Station of sampling	Mechanical analysis of granular material										Silt 0.02 to 0.07 mm. ¹	Clay under 0.02 mm. diameter ¹	Dye adsorption No.	Moisture equivalent	Lower liquid limit	Plastic index	Shrinkage		Depth of topsoil surface at center line	Condition of the topsoil surface
			Over 1/4-inch	1/4-inch-No. 10	No. 10-No. 20	No. 20-No. 60	No. 60-No. 100	No. 100-No. 200	Total	Limit	Ratio											
			Per cent	Per cent	Per cent	Per cent	Per cent	Per cent	Per cent	Per cent	Per cent	Inches										
1528	1	2	3	3	7	31	9	7	60	13	27	13	12	25	7	16	1.8	6	Smooth and well bonded.			
1529	2	22	6	7	9	33	10	10	75	9	17	3	6	16	0	13	1.9	6	Badly broken, poorly bonded.			
1530	3	38	3	4	4	27	13	10	61	13	26	6	10	19	3	14	1.9	7	Smooth and well bonded.			
1531	3	74	1	3	7	33	9	6	59	9	32	11	12	27	10	17	1.8	8	Do.			
1532	3	106	1	4	7	37	11	10	70	8	22	7	7	19	0	15	1.9	9	Good; some loose material on the surface.			
1533	4	125	4	6	7	33	12	11	73	9	18	5	7	18	0	15	1.8	7	Considerable loose material, poorly bonded.			
1534	4	145	8	5	5	26	12	10	66	14	20	5	8	19	0	15	1.9	6	Good, except poorly bonded at the edges.			
1535	5	170	8	7	5	28	12	12	72	12	16	5	6	17	0	14	1.9	8	Very loose, poorly bonded.			
1536	5	213	2	6	10	35	11	8	72	10	18	5	7	19	0	16	1.8	6	Do.			
1537	5	244	7	8	9	29	8	8	69	10	22	5	8	19	4	15	1.8	4	Good; some loose material at sides.			
1538	5	269	2	8	9	30	11	8	68	12	21	6	9	21	0	16	1.8	5	Do.			
1539	6	290	2	6	11	41	13	12	85	8	8	3	4	25	0	20	1.7	8	Very poor, loose and scaling badly in places.			
1540	6	340	3	5	8	32	11	11	70	10	21	5	8	19	0	15	1.8	8	Good, some loose material at sides.			
1542	7	5	4	7	8	26	9	12	66	13	21	7	8	19	0	15	1.8	7	Smooth, well bonded.			
1543	7	28	14	9	8	22	8	10	71	13	17	6	8	19	0	15	1.8	6	Do.			
1545	8	416	8	3	5	34	12	9	71	10	20	5	8	17	0	15	1.8	5	Loose and poorly bonded.			
1546	8	472	5	3	6	37	13	11	75	9	16	3	6	17	0	15	1.7	6	Rough, well bonded.			
1547	8	497	8	4	4	29	9	10	64	9	27	7	13	31	6	21	1.7	6	Loose and poorly bonded.			
1549	9	525	11	5	5	38	14	11	84	8	10	2	4	15	0	15	1.8	6	Good, well bonded.			
1551	10	552	3	4	6	33	10	11	67	10	22	9	10	23	0	19	1.7	5	Loose, poorly bonded.			
1553	10	585	3	2	4	37	13	10	69	9	23	5	8	19	0	15	1.8	4	Good, well bonded.			
1554	10	620	1	4	6	36	11	11	69	9	22	5	9	18	0	15	1.8	6	Good, fairly well bonded.			
1555	11	661	4	5	7	34	11	8	69	9	20	8	8	9	0	16	1.7	8	Do.			
1556	11	693	8	10	7	30	11	9	75	10	16	4	7	18	0	14	1.7	5	Good, well bonded.			
1558	11	726	2	7	7	26	10	12	64	12	26	7	12	23	0	19	1.6	5	Smooth, poorly bonded.			
1559	11	757	8	9	6	25	12	11	71	13	15	5	7	17	0	17	1.7	5	Fairly smooth; fairly well bonded.			
1560	11	785	2	5	6	30	16	13	72	11	17	5	8	20	0	18	1.7	5	Loose, very poorly bonded.			
					6	30	16	13	72	11	17	5	8	20	0	18	1.7			Do.		

SAMPLES TAKEN FROM THE SUBGRADE

1565	7	49	0	1	1	26	9	8	45	11	44	10	21	44	22	27	1.5	-----
1548	8	497	0	2	4	26	8	9	49	9	42	11	21	37	12	26	1.6	-----
1552	10	552	5	4	6	29	10	12	66	12	22	7	13	30	0	23	1.6	-----
1557	11	693	1	5	5	23	7	8	49	13	38	8	14	33	15	21	1.7	-----

¹ Limiting sizes of silt and clay are as defined for use in testing semigravel, topsoil, and sand-clay aggregate for road surfacing, U. S. Dept. Agr. Bul. 1216, p. 16. Percentages of silt and clay are based on the total samples, calculated from the results obtained on the material passing the 10-mesh sieve.

TABLE 4.—*Characteristics of material used for covering on surface treatments*

TOPSOIL ON SECTION 3									
Material No.	Where used, stations	Mechanical analysis of granular material						Silt	Clay
		1/4-inch	No. 10	No. 10-20	No. 20-60	No. 60-100	No. 100-200		
		Per cent	Per cent	Per cent	Per cent	Per cent	Per cent		
1530	34+32 to 55+00.....	3	4	4	27	13	10	13	26
1531		1	3	7	33	9	6	9	32
1541		12	3	5	24	10	9	10	27
TOPSOIL ON SECTION 4									
1534	134+00 to 161+04.....	8	5	5	26	12	10	14	20
TOPSOIL ON SECTION 5									
1538	255+00 to 279+84.....	2	8	9	29	11	8	12	21
SAND ON SECTIONS 3, 4, AND 5									
1542	{ 85+00 to 134+00 { 161+04 to 255+00..... }	1	9	25	54	4	2	2	3
Material No.	Where used	Character of material	Mechanical analysis, amount retained						
			1-inch	3/4-inch	1/2-inch	1/4-inch	No. 10	No. 20	
			Per cent	Per cent	Per cent	Per cent	Per cent	Per cent	
27300	Section 6.....	Biotite gneiss.	11	39	81	98			
26953	Sections 7, 8, 10 and on 9 from station 512 to 517.	Gneissoid granite.	19	33	85	98			
26951	Section 9 from station 517 to 533.	Dolomite.....			11	57	93	99	
26952	Section 11.....	Gneissoid granite.	2	17	49	85			

with a grader, and bituminous material was applied to the surface at the rate of one-half gallon per square yard. This was followed by disk harrowing several times, after which the topsoil at the sides of the road was spread over the treatment with the grader. Second and third applications of bituminous material were made, both similar in amount to the first, and followed in each case by disk harrowing, mixing, and shaping with a grader. In mixing, the procedure was to blade all the material from one side of the road to the other, finally shaping the surface to receive the next application.

After the surface had been thoroughly mixed and shaped it was gone over with a steel drag. This method worked very well for both sections, the surface being completely compacted after a few days of occasional dragging.

Although disking did not prove effective in mixing, it is believed that a small amount of such work is beneficial in bringing about a better bond between the mixture and the base and also in preventing the formation of a rich bituminous mat on the surface. It was found that such a mat, if permitted to form, might later peel off and lead to the formation of potholes.

Experimental sections 3 and 4 were both constructed in the same manner. The topsoil road surface was

swept clean, using a revolving broom, followed by hand brooming. The bituminous material was then added in two applications, the first being made at the rate of one-fifth gallon per square yard. The first application was covered with pulverized topsoil from the shoulders on a portion of these sections, while on the remaining portions a fairly coarse local river sand was used.

The first application was left to penetrate and consolidate for a day or two, after which such portions of the surface as had excess covering were swept. The second application was then made at the rate of 0.18 gallon per square yard and covered with the same material as had been used after the first application. The depth of the surface treatment several days after completion was about one-half inch.



APPLYING PRIMING COAT TO THE TOPSOIL SURFACE



STONE COVERING APPLIED IMMEDIATELY AFTER THE APPLICATION OF THE HOT TOP COAT

A marked difference in the appearance of the surface treatment was apparent between the sections where sand covering and topsoil covering were used, the sanded surface appearing richer and less fragile.

The method used on section 5 was similar to that on sections 3 and 4, except that less sweeping was done on the topsoil surface before the first application of oil. Local river sand was used in covering a portion of this section, and on the remaining portion topsoil from the surface of the shoulders was employed.

A small amount of dragging was done shortly after the completion of the treatment to develop a smooth finished surface, but continuous machine maintenance was not feasible with the thin surface mat formed by the single treatment.

DOUBLE TREATMENT USED ON SIX SECTIONS

The remaining experimental sections, Nos. 6 to 11, inclusive, were all of the double surface-treatment type. The general procedure in making these treatments was to sweep the topsoil surface clean of all loose material and follow with a priming coat of a light tar or oil at the rate of one-fourth gallon per square yard. Where holes and breaks occurred in the topsoil surface, repairs were made after the priming coat had been applied. These repairs usually consisted of filling the depression with loose stone, but in the case of very bad breaks a bituminous cold-patch mixture was used. Portions of sections 6 and 11 required a large amount of such repair work.

The priming coat was permitted to penetrate and partially set up for a day or two, after which a heavy tar or asphalt was applied hot at the rate of one-third

gallon per square yard and covered immediately with crushed stone, spread by hand at the rate of 45 pounds to the square yard. After the crushed stone had been spread the surface was rolled with a 5-ton roller and opened to traffic. For a period of two or three weeks while compaction of the surface was taking place, stone thrown to the sides by traffic was collected and respread in places where bleeding and picking up indicated a deficiency of covering.

The priming material used on sections 8 and 9 was a cut-back asphaltic oil. It was found that this oil was not homogeneous, and great difficulty was experienced in applying it, as the distributor nozzles continually clogged. Only the thinner oil, which appeared to be largely the original cut-back naphtha, penetrated the topsoil, while the heavier material formed a mat on the surface.

TABLE 5.—Tests of tars used on experimental sections

Laboratory number	Original treatment							Re-treatment				
	27157	27159	27153	27154	27198	27199	27201	27683	27706	27769	29071	29072
Section	3 and 6	1 and 4	7 and 11	7	9 and 10	10 and 11	11	3 and 4	7 and 10	7, 9, and 10	3 and 4	1
Location of use, stations	34+32 to 108+24, 279+84 to 361	0+00 to 15+84, 108+24 to 161+04	0+00 to 82+50	0+00 to 82+50	512 to 536, 536 to 622+30	536 to 630, 729 to 791+25	630 to 729	34+32 to 161+04	38+00 to 82+50, 574+00 to 622+00	0+00 to 38+00, 512 to 574	34+32 to 161+04	0+00 to 15+84
Specific gravity, 25° at 25° C	1.177	1.184	1.186	1.23	1.231	1.169	1.158	1.186	1.176	1.145	1.144	1.133
Specific viscosity, Engler, 25° C	8.6	20.4	12.4			48.7	36.7			53.8	67.0	69.9
Float test, 32° C, (seconds)						12.0	11.3	23.7	18.9	11.4	21.7	25.3
Float test, 50° C, (seconds)					167							
Bitumen soluble in CS ₂ —per cent.	91.9	91.4	92.2	88.5	81.8	88.0	89.6	89.7		94.75	94.2	91.1
Free carbon—do.	8.1	8.6	7.7	11.4	18.1	11.8	10.3	10.2	8.8	5.19	5.2	4.7
Inorganic matter insoluble—do.	0.0	0.0	0.1	0.1	0.1	0.2	0.1	0.1	0.1		0.1	0.0
Distillation:												
Water—do.	1.6	1.4	1.2	0.0	0.0	2.0	2.4	1.34	1.36	2.57	0.5	4.1
Distilled to 170° C—do.	2.22	2.20	2.29	0.0	0.0	2.79	4.21				0.61	2.54
170 to 235° C—do.	5.06	6.23	7.07	2.25	2.47	9.36	6.26	9.12	4.83	8.80	8.13	5.48
235 to 270° C—do.	13.91	12.55	10.83	7.10	6.54	10.90	9.98	11.50	12.51	12.76	13.05	15.60
270 to 300° C—do.	5.87	6.52	6.49	3.66	4.73	5.10	5.83	5.03	6.24	6.76	9.26	7.41
Residue—do.	73.21	72.62	73.28	87.15	86.60	71.84	73.79	72.94	75.33	69.04	68.80	68.70
Softening point, residue °C—do.	30	45	34	42	49	40	38	38	30	38	50	48

TABLE 6.—Tests of oil products used on experimental sections

Laboratory number	Original treatment							Re-treatment				
	27158 ¹	27160	27161	27197	27200	27202	27203	27684	27685	27705	27760	29073
Section	6	5	2	8 and 9	8	11	11	2	5 and 6	11	8	5
Location of use, stations	279+84 to 361+00	161+04 to 279+84	15+84 to 34+32	436 to 512, 512 to 536	429+50 to 512	700 to 791+25	622+50 to 700	15+84 to 34+32	161+04 to 353+00	622+50 to 791+25	436+00 to 512+00	161+04 to 279+84
Specific gravity 25° at 25° C	1.011	0.938	0.929	0.950	1.031	1.032	1.031	0.9496	0.9496	0.9473	0.9439	0.955
Flash point, °C	195	105	20	28	235	235	235	100	99	90	30	130
Burning point, °C		140	31	31				160	160	150	55	
Specific viscosity, Engler, 25° C		73.7	127.5	329				228.3	164.7	169.3	84.7	127.2
Specific viscosity, Engler, 40° C		30.5	57.5	117				55.7	48.3	44.19	31.8	42.3
Specific viscosity, Engler, 50° C			48.6									
Specific viscosity, Engler, 100° C	82.8				336	327	332					
Float test, 32° C, seconds	517											
Float test, 50° C, seconds												
Penetration, 25° C., 100 grams, 5 seconds					165	161	164					
Softening point, °C					40	40	40					
Loss, 163° C., 5 hours, 20 grams												
Residue, float test, 32° C, seconds	1.23	13.4	24.7	23.0				9.0	9.5	9.6	32.2	7.1
Residue, float test, 50° C, seconds		Fluid.	1671	4122								
Loss, 163° C., 5 hours, 50 grams		174	200	360								8.0
Residue, float test, 32° C, seconds—per cent.	.64	10.4	22.8	21.3	.17	.14	.16	5.7	6.1	6.1	7.4	4.7
Residue, float test, 50° C, seconds		Fluid.	516	852								
Residue, penetration 25° C., 100 grams, 5 seconds	153		131	237								6.0
Bitumen soluble CS ₂ —per cent.	99.9	99.9	100.0	99.8	99.8	99.8	99.8	100.0	100.0	99.9	100.0	99.9
Organic matter insoluble—do.	.1	.1	.0	.2	.2	.2	.2	.0	.0	.1	.0	.1
Inorganic matter insoluble—do.	.0	.0	.0	.0	.0	.0	.0	.0	.0	.0	.0	.0
Per cent bitumen insoluble in 86° B. naphtha	21.5	12.7	19.3	20.0	23.6	24.2	23.5	13.3	12.3	13.3	17.6	7.6
												.06

¹ Sample No. 27158 contained about 5 per cent adventitious water. Tests made after dehydration at 105° C.

COST DATA KEPT ON VARIOUS SECTIONS

Cost data on the several experimental sections are given in Tables 7 and 8. The costs shown do not include charges for overhead or rental of equipment. The unit cost of constructing experimental sections 1 and 2 is high, but would probably be less on larger sections.

Unit costs of materials and labor used on these sections were as follows:

Oil for cold and hot application at refinery-----	per gallon-----	\$0. 05
Tar for cold and hot application, delivered-----	do-----	. 1281
Stone, 1¼ inches to one-fourth inch, delivered-----	per ton-----	2. 05
Common labor, per day of 10 hours-----		1. 75-1. 90

TABLE 7.—Cost per square yard of various items on sections 1 to 6

Section No.-----	1	2	3	4		5			6
Covering material.-----	Top-soil	Top-soil	Top-soil	Sand	Top-soil	Sand	Top-soil	Sand	1¼-inch stone
Area (square yards)-----	3,516	4,103	11,262	5,148	5,998	5,724	5,520	20,854	17,582
Labor preparing road-way.-----	Cents	Cents	Cents	Cents	Cents	Cents	Cents	Cents	Cents
Heating bituminous material.-----	0. 64	0. 58	1. 23	1. 23	0. 35	0. 35	0. 26	0. 26	0. 88
Applying bituminous material.-----									. 28
Hauling covering material.-----	. 46	. 58	. 32	. 32	. 13	. 13	. 13	. 13	. 56
Applying covering material.-----				. 90		. 49		. 41	1. 10
Tar, delivered.-----	1. 44	. 81	. 40	. 40	. 13	. 13	. 19	. 19	. 39
Oil, cold application.-----	18. 93	7. 61	4. 54	4. 54	5. 19	5. 19	1. 86	1. 86	3. 15
Freight on above.-----		2. 60					. 73	. 73	. 58
Oil, hot application.-----									1. 60
Freight on above.-----									5. 18
Cover material.-----									. 83
Fuel, oil, etc.-----	1. 24	1. 76	. 73	. 73	. 23	. 23	. 54	. 54	. 72
Miscellaneous.-----	. 55	. 68	. 15	. 15	. 30	. 30	. 42	. 42	
Total cost per square yard (cents).-----	23. 30	14. 62	7. 37	8. 27	6. 33	6. 82	4. 13	4. 54	15. 27
Cost per mile, 20 feet wide (dollars).-----	2, 735	1, 715	860	970	745	800	485	535	1, 790

TABLE 8.—Cost per square yard of various items on sections 7 to 11

Section No.-----	7	8	9			10		11
Covering material-----	Stone	Stone	Stone	Chats	Slag	Slag	Stone	Stone
Area (square yards)-----	19,950	16,044	1,056	3,378	633	3,589	14,672	35,625
Labor-----	<i>Cents</i> 2. 74	<i>Cents</i> 2. 74	<i>Cents</i> 2. 74	<i>Cents</i> 2. 74	<i>Cents</i> 2. 74	<i>Cents</i> 2. 74	<i>Cents</i> 2. 74	<i>Cents</i> 2. 74
Tar for cold application-----	2. 823					2. 823	2. 823	2. 823
Freight on above-----	. 059					. 059	. 059	. 059
Tar for hot application-----	5. 76		5. 76	5. 76	5. 76	5. 76	5. 76	
Freight on above-----	. 121		. 121	. 121	. 121	. 121	. 121	
Oil for cold application-----		1. 833	1. 833	1. 833	1. 833			
Freight on above-----		. 686	. 686	. 686	. 686			
Oil for hot application-----		2. 028						2. 028
Freight on above-----		. 760						. 760
1 to ¼-inch stone-----	2. 982	2. 982	2. 982				2. 982	2. 982
Freight on above-----	2. 491	2. 491	2. 491				2. 491	2. 491
Limestone chats-----				4. 07				
Freight on above-----				4. 56				
1 to ¼-inch slag-----					1. 933	1. 933		
Freight on above-----					4. 00	4. 00		
Fuel and steam-----	. 3445	. 3445	. 3445	. 3445	. 3445	. 3445	. 3445	. 3445
Miscellaneous-----	. 5655	. 5655	. 5655	. 5655	. 5655	. 5655	. 5655	. 5655
Total cost per square yard (cents)-----	17. 89	14. 43	17. 52	20. 68	17. 98	18. 45	17. 89	14. 79
Cost per mile, 20 feet wide (dollars)-----	2, 100	1, 695	2, 055	2, 425	2, 110	2, 165	2, 100	1, 735

SERVICE HISTORY OF THE EXPERIMENTAL SECTIONS DESCRIBED

Since the original treatment of these sections in July, 1925, some retreatment has been necessary and all sections have been under patrol maintenance. The

patrol maintenance has consisted largely of repairing breaks in the surface, using the same materials as were used in the original treatment. In patching the double surface-treated sections, good results were obtained with the original hot application materials and also with commercial cold patching products.



TAR MIXED SECTION WITHOUT SEAL COAT SHORTLY AFTER CONSTRUCTION

Most of the retreatments were as had been originally planned. In most cases they were for the purpose of sealing the surface or of thickening and strengthening the original treatment. On several sections, however, the work amounted to reconstruction. Table 9 summarizes the nature and cost of the work done on the experimental sections to June, 1927. A description of this work up to August, 1927, and a survey of the condition of the sections at that time is given below in some detail.

The experiments in Anderson County (secs. 1 to 6) have carried much less traffic than those in Spartanburg County. Counts made on the Anderson County experiments during the months from January to September of 1927 showed an average of 456 vehicles and a maximum of 935 for the 12-hour period from 7 a. m. to 7 p. m.

The traffic on the Spartanburg sections has greatly increased since the experiments were completed. Traffic counts made by the State on week days during five summer months of 1924 before these treatments were made, showed an average of 636 and a maximum of 800 vehicles for the 12-hour period from 7 a. m. to 7 p. m., while a similar count during the same period of 1926 gave an average of 834 and maximum of 1,082 vehicles.

SECTION I

A few months after this section was subjected to traffic, evidence of nonuniform mixing developed. Corrugations denoting excessive richness appeared in some places and in others leanness was indicated by raveling. A light seal-coat treatment consisting of tar of a specific viscosity at 40° C. of 18 to 25 was applied at the rate of one-fifth gallon per square yard and covered with sand. This treatment prevented further raveling but the mixture continued to be highly unstable in certain areas.

This section was maintained through the year 1926 by patching and cutting down corrugations, but in March, 1927, it was apparent that further maintenance of this character would not be justified. The surface was scarified to a depth sufficient to produce a surface

TABLE 9.—Cost per square yard of maintenance of experimental sections, from June, 1925, to June, 1927¹

Original treatments (May and June, 1925)				Re-treatments				Maintenance cost per square yard			Average yearly cost of re-treatments and maintenance during year July, 1926, to July, 1927, per mile (20 feet wide)	Cost of re-treatments and maintenance during year July, 1926, to July, 1927, per mile (20 feet wide)	
Station No.	Type	Bituminous material	Covering	Cost per square yard	Date	Type	Bituminous material and rate of application	Covering	Cost per square yard	Cents	Cents	Cents	Dollars
1	Mixed	{ 1.5 gallons tar No. 27159.		23.30	{ July, 1925 { March, 1927	Surface-treated Mixed, surface-treated.	{ ½ gallon tar No. 27159. { 1.1 gallons tar No. 29072. { ½ gallon tar No. 29072.	Sand	5.30 21.20	2.04	4.20	1.50	2,140
2	do.	1.5 gallons oil No. 27161		14.62	{ November, 1925. { March, 1927	{ Mixed. { Mixed, surface-treated.	{ 1.1 gallons oil No. 27684. { ½ gallon oil No. 29074. { ½ gallon oil No. 29071.	Sand	11.40 14.10	.63	.18	2.04	5,195
3	Surface-treated.	0.38 gallon tar No. 27157	{ Topsoil { Sand	{ 7.37 8.27	{ November, 1925. { March, 1927	{ Surface-treated { do. { do.	{ 0.38 gallon tar No. 27683. { 0.18 gallon tar No. 27683. { 0.40 gallon tar No. 29071. 0.25 gallon tar No. 29071.	do.	6.27 3.29 5.31	2.76	3.61	1.96	2,045
4	do.	0.38 gallon tar No. 27159	{ do. { Topsoil	{ 6.82 6.33	{ November, 1925. { March, 1927	{ do. { do. { do.	{ ½ gallon tar No. 27683. { 0.37 gallon tar No. 27683. ½ gallon tar No. 29071.	do.	3.92 6.22 5.31	4.63	5.03	2.76	1,290
5	do.	0.38 gallon oil No. 27160	{ do. { Sand	{ 4.13 4.54	{ November, 1925. { March, 1927	{ do. { do. { do.	{ ½ gallon oil No. 27685. { ½ gallon oil No. 29073.	do.	2.94 3.06	2.63	2.35	2.33	935
6	Double surface-treated.	{ Primer: ¼ gallon tar No. 27157. { Top coat: ½ gallon asphalt No. 27158.	1 to ¼-inch stone	15.27	November, 1925.	do.	½ gallon oil No. 27684-5	do.	3.22	2.66	1.48	.87	530
7	do.	{ Primer: ¼ gallon tar No. 27153. { Top coat: ½ gallon tar No. 27154.	do.	17.89	{ do. { March, 1926	{ do. { do.	{ Stations 38 to 82+50, ½ gallon tar No. 27706. { Stations 0 to 38, ½ gallon tar No. 27769.	do.	3.49 3.51	1.73	1.35	.70	520
8	do.	{ Primer: ¼ gallon oil No. 27197. { Top coat: ½ gallon asphalt No. 27200.	do.	14.43	do.	do.	½ gallon oil No. 27760.	do.	3.24	1.50	1.13	.70	400
9	do.	{ Primer: ¼ gallon oil No. 27197. { Top coat: ½ gallon tar No. 27198.	{ 512 to 517, 1 to ¼ inch stone. 517 to 533, ¼-inch chats. 533 to 536, 1 to ¼ inch slag.	{ 17.52 20.68 17.98	do.	do.	½ gallon tar No. 27769.	do.	4.75	1.70	.67	.31	465
10	do.	{ Primer: ¼ gallon tar No. 27199. { Top coat: ½ gallon tar No. 27198.	536 to 553, 1 to ¼ inch slag.	18.45	{ November, 1925.	do.	{ Stations 574+50 to 622+50, ½ gallon tar No. 27706. { Stations 536 to 574+50, ½ gallon tar No. 27769.	Sand	3.41 8.91 4.80	1.88	2.12	.89	635
11	do.	{ Primer: ¼ gallon tar No. 27153. { Top coat: ½ gallon asphalt No. 27202.	{ 553 to 622+50, 1 to ¼ inch stone. 1 to ¼ inch stone.	{ 17.89 14.79	March, 1926 November, 1925.	do.	½ gallon oil No. 27705.	{ Stations 622+50 to 720, 735 to 750, sand. Remainder, ¾ to ¾ inch stone.	2.38 4.07	3.64	1.89	1.71	680

¹ Above costs do not include rental or depreciation.

layer $3\frac{1}{2}$ inches in thickness. An application of tar at the rate of 1.1 gallons per square yard was required on account of the greater depth of the new surface layer and extreme care was taken to secure a homogeneous and uniform mixture. After the surface had been exposed to traffic for a few days a seal-coat application of one-fifth gallon of tar per square yard was applied and covered with sand.

The reconstructed section appears to have been very successful. Neither raveling nor evidences of instability have developed. The seal coat exhibited a tendency to peel off, due probably to the blotting action of the fine sand cover which, with the tar, formed a lean mat which did not adhere closely to the mixture beneath.

SECTION 2

The quick-curing oil used on this section produced a more brittle mixture than was desired. It was intended that the surface should possess sufficient plasticity to be self-sealing. However, the mixture proved to be lean except in spots where, due to the nonuniformity of composition, the surface corrugated. Late in the fall of 1925 the entire section was scarified and remixed, the mixture being enriched by the addition of 1.1 gallons per square yard of the slow-curing oil used in the construction of section 5.

Throughout the winter this section remained smooth and well bonded, but with the advent of warm weather, instability developed and the surface rutted and corrugated badly. It was evident that, although satisfactory under winter conditions, this mixture was too rich to withstand traffic during the summer months.

The section was reconstructed in March, 1927, by scarifying and adding 1.1 gallons per square yard of slow-curing oil. Mixing was carried to a depth of about $3\frac{1}{2}$ inches, thus producing a much leaner mixture despite the addition of more oil. As a protection to the surface from the abrasive action of traffic, one-fifth gallon per square yard of the same oil was applied and covered with fine sand.

This surface application acted much the same as that on section 1, peeling off badly within a few weeks and it appeared that resealing would be required in the near future.

SECTIONS 3 AND 4

The portions of these experiments upon which topsoil cover was used originally, broke up badly shortly after construction and were scarified and refinished as topsoil surfaces and left to consolidate for four months and then retreated. The sand-covered portions remained in fair condition.

In pursuance of the original plan to develop a mat of appreciable thickness by subsequent treatments, the entire surface of both experiments was treated with tar of 18 to 25 specific viscosity at 40° C. during the following fall. The areas which had been scarified were given about one-third gallon per square yard and about one-fourth gallon per square yard was applied to the remainder. Sand was used as the mineral cover throughout.

The bituminous mats on these experiments were thin and rather brittle and suffered severely from the steel-tire traffic to which they were subjected. This condition was more prevalent on areas of the road where the topsoil was not well bonded.

In March, 1927, those portions which had given particular trouble, aggregating about one-half mile,

were re-treated with 0.4 gallon of tar per square yard. At the same time the remainder of the sections were more lightly treated with one-fourth gallon per square yard. The tar was of 13 to 18 specific viscosity at 40° C. and sand was used as the cover.

In general, these sections were in good condition. The mat proved stable and as weak spots were gradually corrected the amount of maintenance required to keep them in repair decreased.



TAR PENETRATION TREATMENT ON SECTIONS 3 AND 4. THE THIN BRITTLE SURFACE BROKE AND SCALED BADLY DURING THE FIRST YEAR BUT WAS MATERIALLY STRENGTHENED BY LATER RE-TREATMENTS

SECTION 5

This section was in good condition in November, 1925, although a few breaks had occurred in the oil mat. Following the original plan of thickening this mat by repeated treatments, an application of one-fifth gallon of the original slow-curing oil per square yard was made and covered with sand. The surface remained generally in good condition during the summer and winter of 1926. Minor breaks developed early in 1927 and during March of that year the section was again re-treated as before. At the time of inspection the section was in excellent condition. It appeared likely that little maintenance would be required for some time.

SECTION 6

A large portion of the topsoil surface on this section was poorly bonded. In such places the surface treatment failed in many instances and made a large amount of patching necessary during the first few months.

For the purpose of strengthening and resealing the mat, a treatment of one-fifth gallon per square yard of slow-drying oil and sand was applied in November, 1925, to the whole section with the exception of 800 feet at the west end. The untreated portion was in relatively good condition and it was desired to observe its behavior during the winter.

Numerous breaks developed in the surface mat along the edges during the summer of 1926, and were undoubtedly due to poor support by the topsoil. The repair of these defects and subsequent ordinary maintenance has kept the experiment in good condition and when last inspected no further general re-treatment was thought to be necessary in the near future.

SECTION 7

The first 3,800 feet of this section was in excellent condition late in the fall of 1925 and was permitted to go through the winter without re-treatment. The remaining portion of the surface had developed numer-

ous breaks and was therefore given an application of tar of 18 to 25 specific viscosity at 40° C. at the rate of one-fifth gallon per square yard. This material was applied cold and was covered with sand.

The latter portion remained in excellent condition during the winter but considerable patching was required on the part upon which the fall re-treatment had been omitted. In March of 1926 this area was given an application of tar at the rate of one-fifth gallon per square yard and covered with sand.

The condition of this experiment has since continued entirely satisfactory.

SECTION 8

This section was not given a re-treatment in the fall of 1925 with the result that a large amount of patching was required during the winter and early spring.

A re-treatment consisting of one-fifth gallon of a medium quick-curing oil per square yard with a covering of sand, was applied during March, 1926. The surface has since continued in excellent condition and, when last inspected, was not regarded as likely to require early re-treatment.

SECTION 9

The surface of this section was smooth and showed no defects in the fall of 1925. The portion covered with dolomite chats had an appearance resembling a Topeka pavement, while that on which coarse slag was used possessed a rough and granular texture.

Although some patching was required during the first winter after construction on the portion covered with chats, its condition was, in general, much better than that on which coarse slag was used.

The entire section was given a re-treatment of tar of 8 to 13 specific viscosity at 40° C. and a sand cover during March, 1926. The re-treatment resulted in marked improvement of the surface, and its subsequent behavior has been entirely normal and satisfactory.

SECTION 10

The surface between stations 574+50 and 622+50 was treated during the late fall after construction with one-fifth gallon of tar of 13 to 18 specific viscosity at 40° C. and covered with sand. All of this section was not treated as it was desired to retain the rough, granular surface of the slag portion over the winter and at the same time secure information as to the need for the early re-treatment.

The portion of the surface which had not been treated in the fall developed many breaks during the winter. These breaks were repaired during March, 1926, and the surface sealed with an application similar to that applied to the other portion during the fall. The entire section was in good condition at the time of inspection.

SECTION 11

The surface of the north half of this section broke up badly during the first year after construction. On the remaining portion where the underlying topsoil was relatively well bonded the surface was in much better condition.

A surface treatment was applied during November, 1925, using a slow-drying road oil. The oil was covered with sand except for two short sections upon which stone chips were used.

The surface of this experiment has continued in good condition except for a stretch of about 1 mile on which the topsoil was loosely bonded and unsuitable for this type of construction.

EXPERIENCE INDICATES IMPROVEMENTS IN METHODS OF DOUBLE-SURFACE TREATMENT

Observation of the behavior of the different sections since the time of construction has suggested several changes in the character of the bituminous materials and methods of construction, which it is believed would improve the surface. In a few cases such changes have been carried out in reconstruction. The following discussion touches on such factors as appear from the results of these experiments to have an important effect on the results secured with the different treatments.



A WELL BONDED TOPSOIL SURFACE SUITABLE FOR DOUBLE SURFACE TREATMENT

A well-bonded surface with good support is essential for the double-surface treatment of the kind used on sections 6 to 11, inclusive. Local surface failures which have occurred in these experiments, developed where the topsoil surface was poorly bonded. Such areas may, of course, be anticipated in any topsoil road, but a period of constant and patient maintenance will generally result in a satisfactory bituminous surface.

A priming coat seems to be desirable for this type of construction in order to secure a good bond between the surface bituminous mat and the topsoil, although on a short portion of section 6 where the priming coat was omitted, the surface is still in perfect condition. On this area, however, the application of hot material was heavier than the rest and a thicker mat was originally formed. Tar of 8 to 13 specific viscosity at 40° C. was used as primer. On well-bonded topsoil surfaces the penetration obtained ranged from one-quarter to three-eighths-inch. Greater penetration seemed to be obtained when the topsoil surface was slightly moist than when it was dry and somewhat dusty. Both tar and asphaltic oil were used for binder, and one-third gallon per square yard seemed to be about the correct amount. In one case shoving developed where 0.45 gallon per square yard was used. Both tar and asphaltic oil seem to be equally satisfactory in producing a well-bonded topsoil surface.

A fragile granite ranging from 1/4 to 1 1/4 inches in size was used for covering on most of the double surface-treatment experiments. This stone crumbled to a much smaller size under rolling and early traffic. Where a hard, tough slag of the same size was used the surface remained open and rough. Evidently the size used was too large for this type of surface treatment.

except in the case of the stone which broke up readily. For a stone of fairly tough character, $\frac{1}{4}$ to $\frac{3}{4}$ inch in size should prove more suitable.

A supplementary treatment which will seal any small breaks or areas not sufficiently covered by the first application of heavy material has been shown to be desirable before the original treatment goes through the first winter. On several areas where this treatment was omitted, increased maintenance was needed.

Further experimental work will be required to determine the types of materials most suited for supplementary treatments. Two grades of road oil, slow-drying and quick-drying, respectively, and refined tar of 8 to 13 and 18 to 25 specific viscosity at 40° C. were used for this purpose. In some cases the sections re-treated with a cold application of oil showed somewhat uneven covering, due probably to the shearing action of traffic on the thin soft mat before it had bonded firmly. In the case of the slow-curing oil, brooming the sand covering occasionally for a few days following the treatment seems to aid in securing a more uniform surface mat. Of the two tars, the heavier seems to have given a better seal. It is possible that too much covering material was used for these light bituminous materials.

The purpose of the original seal coat is primarily to seal the surface, while later treatments may be required for this purpose and also to increase the plasticity of the bituminous mat and to add to its thickness. It seems that such results might best be obtained with an occasional light application of a heavier grade of tar or a cut-back oil prepared from an adhesive asphalt base.

The character of the mineral cover is as important as the bituminous material. A clean, coarse sand or stone or slag chips not exceeding one-half inch in maximum size is probably best suited for the purpose.



AN EXAMPLE OF EXCESSIVE SCALING DUE TO THE IMPROPER SELECTION OF SURFACE TREATMENT MATERIALS

Sparing use of coarse sand is preferable if the object is to reseal or enrich the surface. Coarser material applied more heavily should be used in thickening or correcting inequalities in the earlier treatments. The above illustration of similar work on a neighboring project shows an extreme case where materials selected for re-treatments were not suitable for use on the hard smooth surface of the original treatment.

MAINTENANCE AN IMPORTANT FACTOR

The double-treated sections have received only one re-treatment which was applied primarily to seal the surface. On a few of the experiments this was applied

before the first winter, while on others re-treatment was not made until a year after the original construction. The question of how often such treatments will be required for the purpose of sealing and adding life and thickness to the bituminous mat has not been answered. South Carolina has several miles of this type of surface which have been in service for three years and have received only the first re-treatment. It is possible that a re-treatment as often as every other year may prove economical by keeping the patrol maintenance cost at a minimum.



APPEARANCE OF DOUBLE SURFACE TREATMENT SHORTLY AFTER CONSTRUCTION. PATROL MAINTENANCE NEEDED AT THIS STAGE TO RESPREAD LOOSE COVERING AND REPAIR BROKEN PLACES

The degree of success to be obtained by this method depends almost as much upon correct maintenance as upon the original treatments. The careful attention devoted to details of construction must not be relaxed in maintenance if the greatest benefits are to be derived.

Patrol maintenance was found to be necessary immediately after the treated surfaces were opened to traffic. It is very important to repair early breaks and respread such stone as may be displaced under traffic. The cost of such maintenance is high during the first six months but thereafter decreases greatly.

A large portion of the patrol maintenance consisted of repairing breaks occurring along the edges. Such breaks developed frequently at places where the topsoil had not been well bonded previous to applying the surface treatment. A substantial reduction in the maintenance cost on surface treatments of this type will result if some method can be devised for strengthening this portion of the surface. The South Carolina highway department is at present experimenting with various methods of edge thickening. However, until a more adequate method is developed, good results can be expected from widening the surface and treating to a greater width.

Maintenance costs on the 3-year-old double surface-treatment experiments of the State highway department indicate the decided economy of such treatments in comparison with untreated surfaces, which for the same traffic conditions cost on the average from \$500 to \$600 per mile per year. The present work was, of course, relatively expensive during the year immediately following construction, but during the last six months the costs have been low, substantiating the conclusion that with proper attention to maintenance ultimate economy may be confidently expected.

Based on the successful results obtained with this type of treatment, South Carolina has recently treated a considerable mileage of well-bonded topsoil roads with double surface treatment.

ENCOURAGING RESULTS SECURED WITH MIXING METHOD ON POORLY BONDED SURFACES

A large mileage of the topsoil roads has poorly bonded surfaces. It was hoped to develop a mixing treatment suited to this condition. At first it was thought that the mixture should be sufficiently rich in bitumen to seal itself under traffic. However, experiments based upon this theory proved unsatisfactory because of the shoving that developed. In the reconstruction of such sections a lean mixture was used and a seal coat added. The indications are that his latter construction gives a more stable mixture protected by a richer and better sealed surface.



A TOPSOIL SURFACE LOOSELY BONDED AND NOT SUITABLE FOR DOUBLE SURFACE TREATMENT. A SLOW-DRYING OIL PENETRATION OR MIXED TREATMENT WOULD BE BETTER FOR THIS CONDITION

Tests made on the original mixture taken from the surface showed from 5 to 7 per cent of bituminous binder. Shoving had taken place where the binder occurred in excess of 6 per cent. With the type of soil which constituted the surfacing of the mixed sections, about 1.5 gallons per square yard of nonvolatile binder is required to produce a stable mixture of 2-inch thickness. This quantity of bitumen was insufficient to thoroughly coat the grains of the clay-bearing aggregate used on the mixed sections and the bond was derived in part from the clay. Other experiments have shown that in the absence of clay a sufficient amount of bituminous binder must be provided to entirely coat the particles. Relatively clean, very fine sandy soils may require as high as 10 or 11 per cent by weight of bitumen.

These amounts are, of course, exclusive of the bituminous material required for surface enrichment or seal coating. The body of the base mixture is fragile and easily abraded and must be protected against the action of traffic and weather. In case a highly volatile cut-back bituminous product is used in the mixture, an excess must be provided to compensate for the loss of fluxing material through volatilization.

Refined tar of 18 to 25 specific viscosity at 40° C. was used on section 1. On section 2 a quick-drying road oil was used in the original treatment and a slow-drying oil in the later re-treatment. The tar and quick-drying oil behaved similarly, both producing mixtures which soon became brittle. The slow-drying oil was substituted in the re-treatment of section 2 in order to adapt it to machine maintenance.

Although no particular difficulties were experienced in the construction of these sections, care is needed in mixing and compacting to prevent the formation of rich and lean layers, which tend to occur when compaction of the full depth is attempted in one operation. This tendency was largely overcome by progressively building up the surface layer. After spreading the mixture to the full depth, all except a thin layer was bladed off. Again the material was spread over the road and again it was removed, leaving an added thickness above that which first remained. This process was repeated until the full depth was attained.

The results obtained to date on these two sections do not conclusively establish the value of this type of treatment but the indications are encouraging where the lean mixture and surface enrichment method has been used.

COLD SURFACE TREATMENT WITH SLOW-DRYING OIL SUITED FOR SPECIAL CONDITIONS

Sections 3 and 4 were treated originally with tar of 8 to 13 and 18 to 25 specific viscosity at 40° C., respectively. In the later treatment both experiments were combined, using the heavier tar. These sections have required a large amount of maintenance. The thin mat is brittle and breaks very often, particularly under steel-tire traffic. Both sections are in fairly good condition at present and will probably require much less attention in the future. However, future possibilities of this type of treatment can not be considered entirely satisfactory in view of the accumulated maintenance costs.

Section 5 is similar to sections 3 and 4, except that in this section a slow-drying oil was used. It was hoped that by using this material machine maintenance could be used. This was found to be impossible without breaking the thin surface mat. The present condition of the section is very good. The built-up mat is plastic and does not break as readily as that containing tar. Considering the present condition, together with the cost to date, this treatment can be regarded as satisfactory for use on roads carrying light traffic or as a temporary expedient on heavy-traffic routes.

TESTS WARRANT CERTAIN CONCLUSIONS

Results obtained with the different experimental treatments up to this time seem to warrant the following general conclusions:

A double surface treatment composed of a prime coat followed with a binder coat of heavy bituminous material and mineral covering is a successful and economical treatment for soil-type roads, provided the surface is well supported and bonded.

Surface treatments of well bonded topsoil roads with light tar are not likely to be successful, as a thin fragile mat is formed which breaks excessively under traffic.

Surface treatments of well-bonded topsoil roads with light slow-drying oil, which forms a plastic mat, are likely to prove successful under light traffic or, for a limited period, under heavier traffic.

Results on mixed treatment experiments are inconclusive, but indicate the probable success of lean mixtures supplemented by surface enrichment.

A light supplementary application of bituminous material which will enrich the surface and at the same time seal any small breaks or areas not sufficiently covered has been found to be necessary shortly after the original treatment.

THE ACTION OF SULPHATE WATER ON CONCRETE¹

FURTHER TESTS OF SPECIMENS IMMERSSED IN MEDICINE LAKE, S. DAK.

Reported by DALTON G. MILLER, Drainage Engineer, U. S. Bureau of Public Roads

Tests on concrete cylinders immersed in the sulphate waters of Medicine Lake, S. Dak., for periods up to three and one-half years have yielded considerable additional data to that contained in the report on the tests made at the end of one year's immersion.² As pointed out in the earlier report, these experiments were designed principally to aid in the general improvement of farm drintile and particularly to develop tile that will endure under the wide range of soil conditions peculiar to Minnesota. The results are applicable, however, to many other sections of the United States and to concrete culvert pipe subjected to the action of sulphate waters under conditions similar to those to which drintile are subjected.

In carrying out the field tests, several hundred concrete cylinders of various materials and treatments were immersed in Medicine Lake, S. Dak. The laboratory routine of making the cylinders and the conditions of exposure were discussed in the first report so that only such general details will be reviewed as are necessary to make possible a clear understanding of the data presented.

TEST CYLINDERS DESCRIBED

Each series of test cylinders consisted of five batches made on different days, and nine cylinders were made from each batch. All cylinders were 2 by 4 inches in size. The concrete cylinders were mixed in the proportion 1:3 by volume with a relative consistency of 1.00 and water ratio of about 0.62. The aggregate passed all standard physical tests and was separated into screen sizes and recombined for each batch to produce a fineness modulus of 4.67. The mortar cylinders were made of standard Ottawa sand as representing fairly well a poorly graded aggregate such as is too often used in smaller size tile.

Extreme care was exercised to make the cylinders of a series as nearly uniform as possible and variables were introduced singly in the different series so that the influence of any variable on the life of cylinders of a series might definitely be determined.

Thirteen different materials were used as admixtures in cylinders. These materials were as follows: Akagel "A," barium chloride, blast-furnace slag, Cal, calcium chloride, Celite, Colloy, Ironite, powdered fuel ash, Trass, Truscon, sulphur, and volcanic ash. The amounts of these materials used expressed as a percentage of the weight of cement used are shown in Tables 2 and 3. Specimens were also surface treated with Inertol, linseed oil, and sulphur.

Special curing conditions included curing in water vapor and steam ranging in temperature from 70° to 285° F. and covered a wide range of variables as to the time of application and duration of the special curing period.

All of the cylinders for exposure in the lake were numbered and placed in copper-nailed wooden crates with a capacity of 50 cylinders each. In all of the crates the cylinders in any vertical row represent the same series. The crates were installed in the lake in about 5 feet of water and were not subjected to freezing and thawing.

MEDICINE LAKE DESCRIBED

Medicine lake is a natural alkali lake located 18 miles northwest of Watertown, S. Dak. It has stretches of gravel beach, so the conditions for installing field specimens are almost ideal. Analyses of water samples taken at different seasons of the year have shown a total salt content ranging between 2.34 and 4.72 per cent, consisting almost entirely of a combination of magnesium and sodium sulphates in which the magnesium salt greatly predominates. (See Table 1.) These are the salts that have caused trouble in Minnesota. The larger of these two figures represents a total salt content somewhat in excess of any soil water so far found in contact with tile in Minnesota, and much more severe than is ordinarily encountered in agricultural lands anywhere.

TABLE 1.—Analyses of water from Medicine Lake, S. Dak.

[Analyses by the water and beverage laboratory, Bureau of Chemistry, U. S. Department of Agriculture]

No.	Date taken	Radicals								
		Na (Calc.)	Ca	Mg	NO ₃	Cl	SO ₄	CO ₃	HCO ₃	Total
MILLIGRAMS PER LITER (PARTS PER 1,000,000)										
778.2..	Dec. 10, 1923	3,755	1,006	5,012	1	580	28,898	84	282	39,618
781.2..	Feb. 14, 1924	4,344	1,082	6,116	Trace.	596	34,533	54	490	47,215
803.2..	Apr. 29, 1924	395	344	4,276	Trace.	396	17,585	98	323	23,417
808.2..	July 1, 1925	3,649	436	4,910	Trace.	464	27,067	114	158	36,828
PERCENTAGE REACTING VALUES										
778.2..	-----	13.05	4.01	32.94	0.00	1.31	48.10	0.22	0.37	100.00
781.2..	-----	12.67	3.62	33.71	.00	1.13	48.22	.12	.53	100.00
803.2..	-----	2.23	2.22	45.55	1.00	.45	47.14	.42	.69	100.00
808.2..	-----	13.58	1.86	34.56	.00	1.12	48.25	.41	.22	100.00

For comparative purposes, specimens similar to those immersed in the lake were stored in a tank of ordinary tap water at the laboratory.

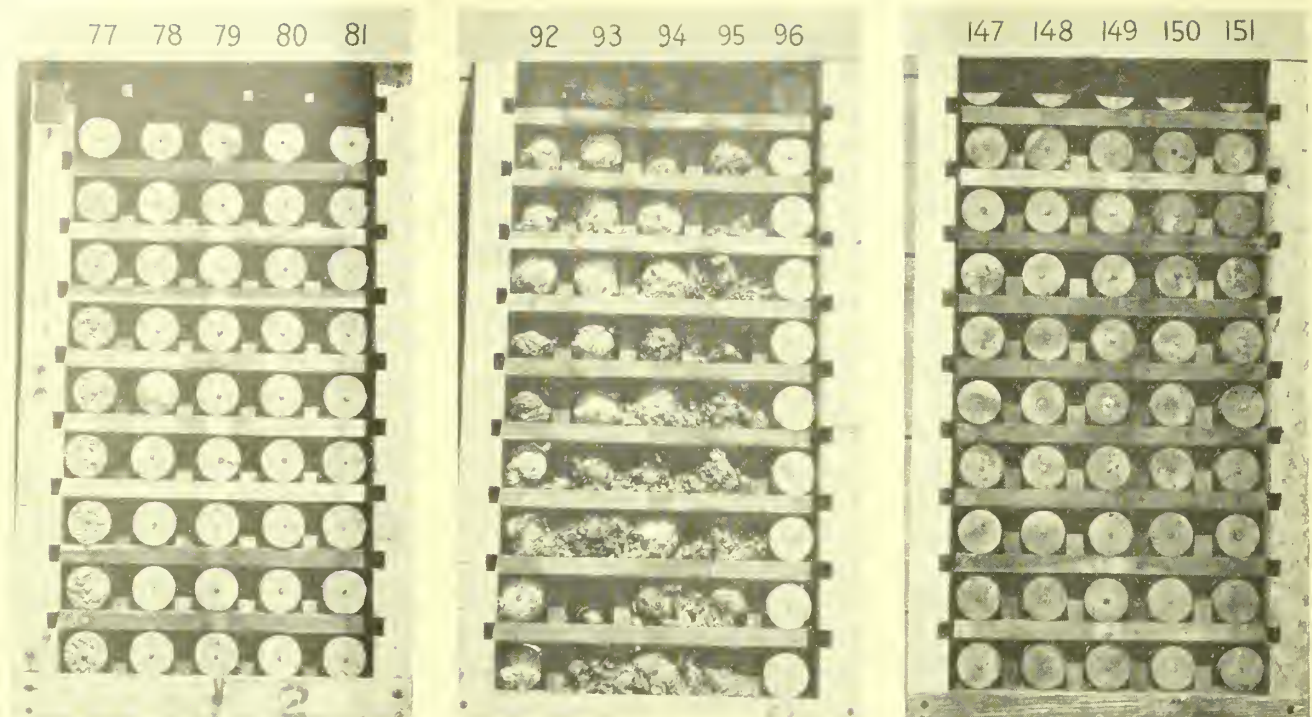
Tests have been made on cylinders at periods ranging from one to three and one-half years. Figures 1 to 8 show the condition of the various test groups on removal from the lake. Tables 2 and 3 give data on the character, curing, and results of compression tests on the cylinders for both the concrete and mortar specimens.

CONCLUSIONS

The time of exposure of the cylinders has been too short to justify drawing definite conclusions for those that as yet show little evidence of deterioration. However, study of the data brings out several facts, the more important of which are stated below.

¹ This report is the result of experiments at the Drintile Laboratory at University Farm, St. Paul, Minn., conducted by the department of agriculture of the University of Minnesota, the Department of Drainage and Waters of the State of Minnesota, and the U. S. Department of Agriculture. This report has been published under the title "Tests of Concrete Exposed to the Sulfate Waters of Medicine Lake, South Dakota" as University of Minnesota Paper No. 726, Journal Series.

² THE ACTION OF SULPHATE WATER ON CONCRETE, Public Roads, vol. 6, No. 8, Oct., 1925.



Cylinders cured in steam at 212° F. after three and one-half years in lake

Cylinders variously cured after three and one-half years in lake. Series 96 was the only one in which the curing temperature reached 212° F.

Cylinders made with high alumina cement A and variously cured after three years in lake

FIG. 1.—CYLINDERS OF VARIOUS SERIES AFTER THREE AND THREE AND ONE-HALF YEARS IN MEDICINE LAKE. ALL THE CYLINDERS IN ANY VERTICAL ROW ARE OF THE SAME SERIES AND THE SERIES NUMBER IS SHOWN AT THE TOP

(1) Standard Portland cements from different manufacturing plants vary greatly in resistance. This is evidenced by tests of more than 30 different cements after exposure for one year, and in the case of the 4 cements used in series 257 to 260 it will be noted that after two years exposure the cylinders of series 260 still had an average compressive strength of 4,390 pounds per square inch as compared with strengths of 690, 0, and 0 pounds per square inch for series 257, 258, and 259, respectively. (See Table 2 and fig 4.)

(2) All cylinders of high alumina cements made very satisfactory showings, up to and including three years' exposure, with the exception of the mortar cylinders of high alumina cement B and Ottawa sand mixed in the proportion of 1:5. These cylinders, designated as series 280, had almost completely failed at one year. (Fig. 4.) On the other hand the 1:5 high alumina cement B concrete cylinders of series 290 had a strength of 122 per cent of normal at one year and showed no visible disintegration. (Fig. 5.) The natural inference is that high alumina cements, while more resistant than standard Portland cements, are not of such high resistance as will permit of the use of extremely lean mixes for concrete subject to the action of sulphate waters.

(3) Specimens cured in steam at a temperature of 212° F. continue to make excellent showings in all

series, and, without exception, cylinders so cured in series 77, 78, 79, 80, 81, 105, and 106 averaged stronger after three years in the lake than they did at one year. (See Table 2 and fig. 1.)

(4) Cylinders cured in steam at a temperature of 212° F. made equally favorable showings irrespective of brand of cement used in the mix.

(5) There is slight difference between the specimens cured in steam at a temperature of 212° F. and those cured at higher temperatures up to 285°. The left photograph of Figure 6 shows cylinders of series 351 to 355, which taken in numerical order, were cured in temperatures of 190°, 212°, 235°, 260°, and 285°. The fifth cylinder from the top of series 352 shows slight deterioration, while no deterioration is evident in series 353 to 355.

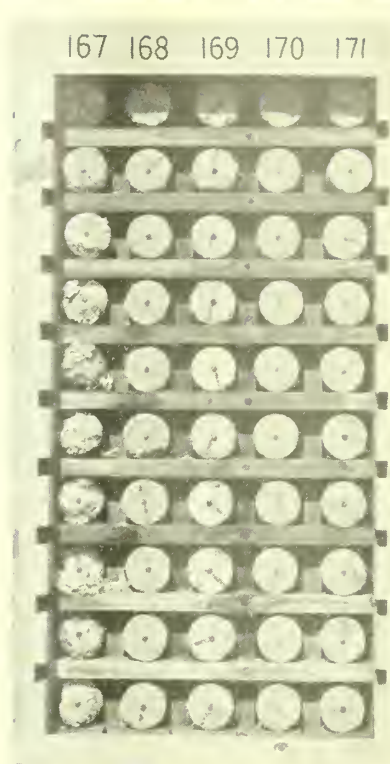
(6) The results secured with admixtures were somewhat indefinite as they were at the end of the first year, but they apparently indicate that certain admixtures, if properly handled, may have sufficient value in developing resistance to justify their use under special conditions. In this connection the cylinders in series 161, 166, 170, 176, and 180 were of greater average strength after two years in the lake than those in the lake or in the laboratory tank at one year. Figures 2 and 3 show the generally excellent appearance of cylinders containing admixtures.



Cylinders with 10 per cent and 40 per cent admixtures of blast-furnace slag, variously cured

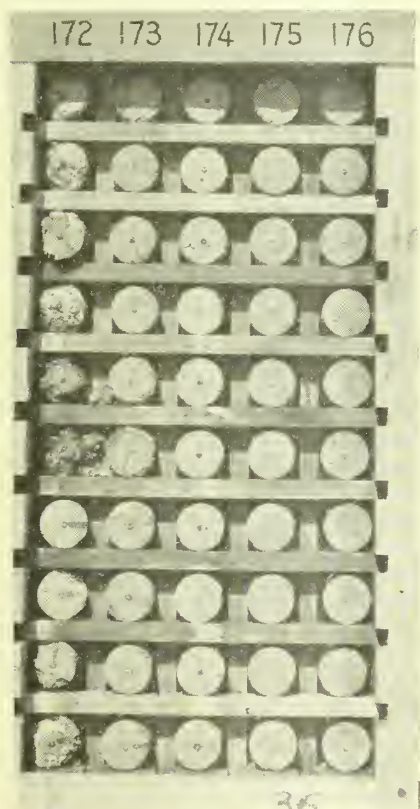


Cylinders with 4 per cent and 8 per cent admixtures of calcium chloride, variously cured

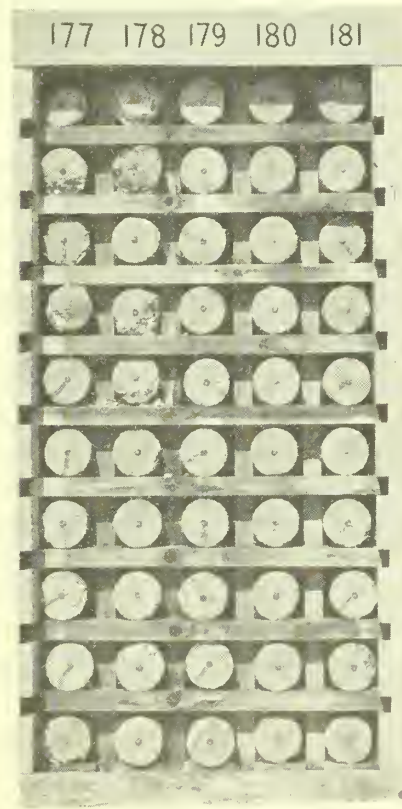


Cylinders with 4 per cent and 8 per cent admixtures of Cal, variously cured

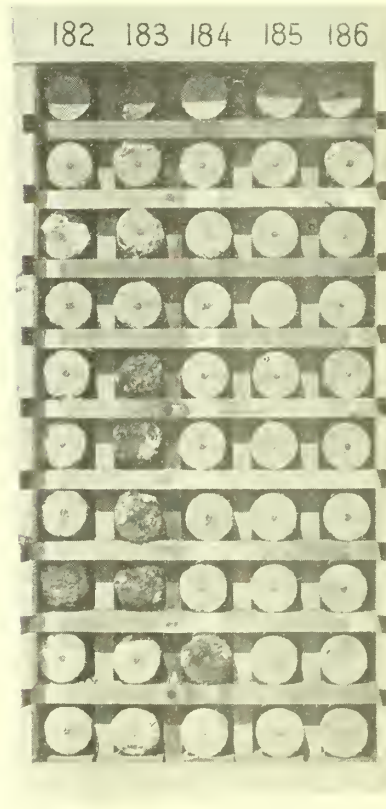
FIG. 2.—CYLINDERS CONTAINING ADMIXTURES AFTER THREE YEARS IN LAKE



Cylinders without admixture and with 20 per cent Ironite admixed and variously cured

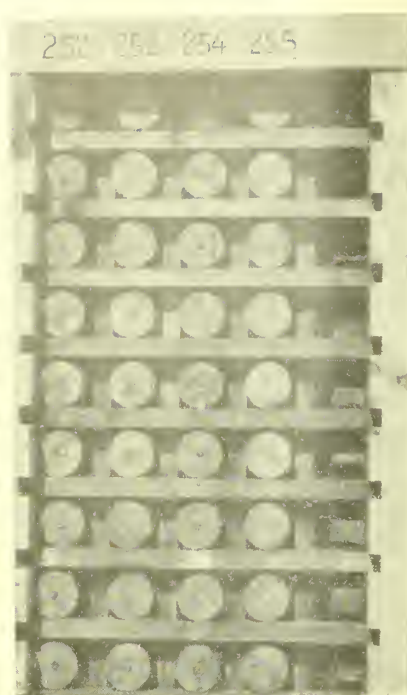


Cylinders without admixture and with 20 per cent volcanic ash admixed and variously cured

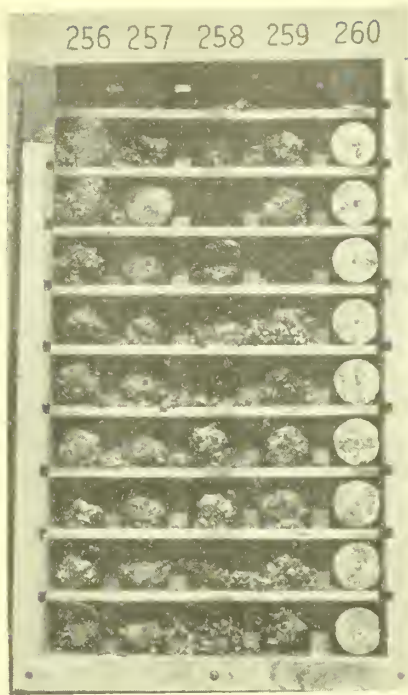


Cylinders without admixture and with 3 per cent Alkagel "A" admixed and variously cured

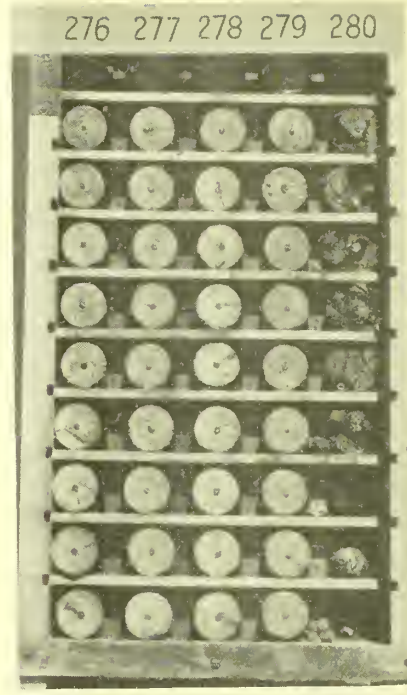
FIG. 3.—COMPARISON OF CYLINDERS WITH AND WITHOUT ADMIXTURES AFTER THREE YEARS IN LAKE



Cylinders of high alumina cement C



Cylinders of different brands of standard Portland cement



Ottawa sand mortar cylinders of various mixes of high alumina cement B

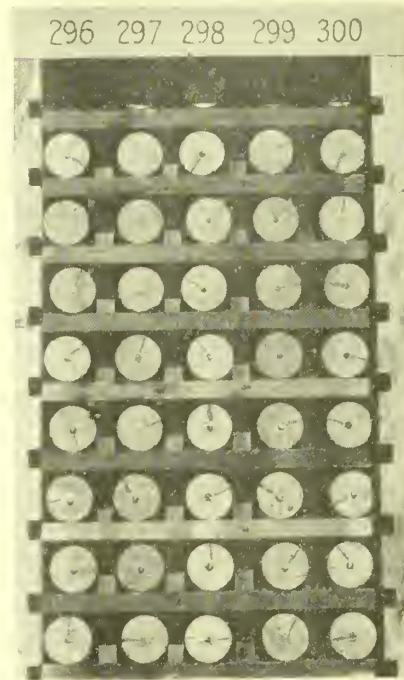
FIG. 4.—COMPARISON OF CYLINDERS OF VARIOUS CEMENTS AFTER TWO YEARS IN LAKE



Cylinders of various mixes of high alumina cement B.



Series 291.—High alumina, cement C, 1:5 mix concrete.
 Series 292.—High alumina, cement C, 1:5 mix mortar.
 Series 293.—Portland cement mortar.
 Series 294.—Portland cement mortar, sulphur impregnated.
 Series 295.—Portland cement mortar, sulphur admixed.



Cylinders cured in water at 212° F. for 48 hours.

FIG. 5.—CYLINDERS OF VARIOUS TYPES AFTER TWO YEARS IN LAKE

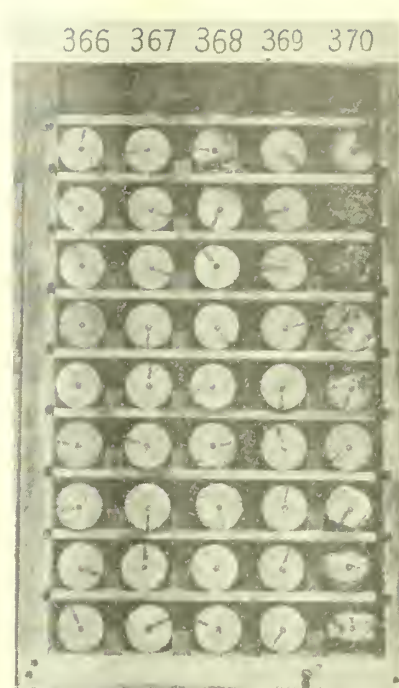


Cylinders cured in water vapor for 12 hours at the following temperatures:

Series 351.—190° F.
 Series 352.—212° F.
 Series 353.—235° F.
 Series 354.—260° F.
 Series 355.—285° F.

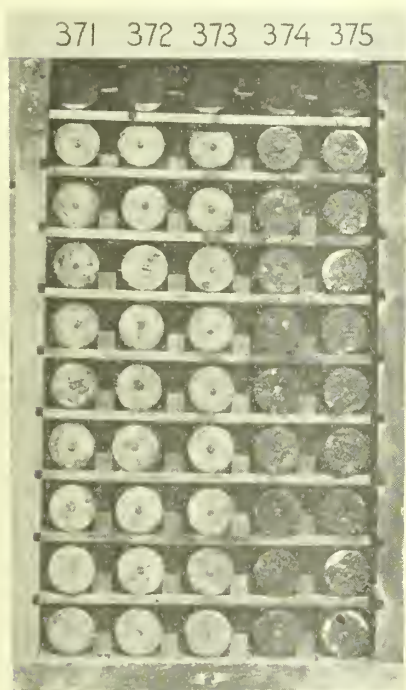


Cylinders made with different brands of standard Portland cement.



Cylinders made with different brands of standard Portland cement.

FIG. 6.—CYLINDERS OF VARIOUS TYPES AFTER ONE AND ONE-HALF YEARS IN LAKE



Cylinders of series 371 to 373 are of standard Portland cement. Those of 374 and 375 are concrete and mortar cylinders, respectively, treated with Inertol.



Cylinders containing admixtures of Trass ranging from zero on the left to 66 per cent on the right.



Cylinders made with different brands of standard Portland cement.

FIG. 7.—CYLINDERS OF VARIOUS TYPES AFTER ONE YEAR IN LAKE

TABLE 2.-Description and tests of all laboratory standard concrete cylinders immersed in Medicine Lake, S. Dak., and parallel tests on specimens stored in laboratory tanks

[Unless otherwise noted the fineness modulus of aggregate is 4.67 and the mix is 1 : 3. Each test result, with a few exceptions, is an average of five cylinders made on different days]

Series No.	Cement laboratory No.	Cement	Water ratio	Admixture, impregnation or surface treatment	Curing method						Average of compression tests							
					Time in moist closet	Time in water	Time in water, vapor or steam	Temperature of water, vapor or steam	Time in air	Absorption at 21 days	Tank specimens				Percentage of normal strength as indicated by tank specimens at one year	Lake specimens		
											7 days	28 days	1 year	1 year		2 years ¹	3 years ¹	
Hours	Days	Hours	° F.	Days	Per cent	Lbs. per sq. in.	Lbs. per sq. in.	Lbs. per sq. in.	Lbs. per sq. in.	Lbs. per sq. in.	Lbs. per sq. in.	Lbs. per sq. in.						
77	11	One-half Portland A, one-half Portland B.	0.59		3		69	212	25	7.9	1,110	1,250	1,250	1,210	97		1,340	
78	11	do.	.59		6		66	212	25	7.2	2,260	3,040	3,130	2,840	91		2,930	
79	11	do.	.59		12		60	212	25	6.9	3,400	4,120	4,550	4,410	97		4,940	
80	11	do.	.59		24		48	212	25	6.9	3,060	4,020	4,600	4,330	94		4,860	
81	11	do.	.59		48		24	212	25	6.7	2,910	3,850	4,630	4,410	95		5,340	
82	11	do.	.59		3		69	155	25	6.9	3,360	4,210	5,380	500	9	0	0	
83	11	do.	.59		6		66	155	25	6.9	3,840	4,720	5,700	490	9	0	0	
84	11	do.	.59		12		60	155	25	6.7	4,440	4,960	5,250	1,030	20	0	0	
85	11	do.	.59		24		48	155	25	6.6	4,350	4,550	6,460	1,220	19	0	0	
86	11	do.	.59		48		24	155	25	6.7	3,540	4,210	6,250	1,660	27	0	0	
87	11	do.	.59		3		69	100	25	6.6	3,730	4,120	5,110	1,430	28	0	0	
88	11	do.	.59		6		66	100	25	6.7	3,590	4,320	4,960	1,370	28	0	0	
89	11	do.	.59		12		60	100	25	6.7	4,050	4,680	5,270	1,290	25	0	0	
90	11	do.	.59		24		48	100	25	6.7	4,080	4,220	5,240	1,630	31	0	0	
91	11	do.	.59		48		24	100	25	6.8	3,510	4,060	5,010	1,920	38	0	0	
92	11	do.	.59		24	27			28	6.7	2,620	4,540	5,690	3,780	66	2,270		
93	11	do.	.59		72				25	7.0	3,380	3,560	5,120	3,150	62	1,200		
94	11	do.	.59		24		48	100	25	6.8	3,810	4,090	5,640	2,020	36	560	0	
95	11	do.	.59		24		48	155	25	6.4	4,240	4,370	5,510	1,660	30	690	0	
96	11	do.	.59		24		48	212	25	6.5	3,770	4,410	4,560	4,300	94	4,910		
97	11.8	do.	.59		24		48	155	25	6.3	3,900	4,770	5,690	1,880	33	890	0	
98	11.8	do.	.59		24		{ 24 155 } 24 212		25	6.7	4,180	4,120	5,020	4,340	87	6,030		
99	11.8	do.	.59		24		{ 72 155 } 48 155		24	6.4	4,490	4,830	5,410	1,530	28	560	0	
100	11.8	do.	.59		24		{ 24 212 } 24 155		24	6.2	4,310	4,960	5,730	5,540	97	5,770		
101	11.8	do.	.59		24		{ 24 155 } 48 212		24	6.3	4,110	4,710	4,890	4,640	95	5,970		
102	17	do.	.59		24		{ 48 155 } 24 212		25	6.2	4,550	4,730	5,580	980	18	0	0	
103	17	do.	.59		24		{ 24 155 } 24 212		25	6.0	4,600	5,510	5,290	4,980	94		5,510	
104	17	do.	.59		24		{ 72 155 } 48 155		24	6.1	4,910	5,180	5,180	1,270	25	0	0	
105	17	do.	.59		24		{ 24 212 } 24 155		24	6.0	4,820	5,070	4,910	5,220	106		6,000	
106	17	do.	.59		24		{ 24 155 } 48 212		24	5.7	5,110	5,990	5,320	5,250	99		5,830	
137	14	High alumina A.	.64		24		48	155	25	6.2	3,250	3,550	2,830	3,400	120		4,510	
138	14	do.	.64		24		{ 24 155 } 24 212		25	6.2	3,150	3,670	2,860	3,320	116			
139	14	do.	.61		24		72	155	21	6.1	3,160	3,840	2,640	3,790	144			
140	14	do.	.61		24		{ 48 155 } 24 212		24	6.1	3,190	3,890	2,730	3,790	139			
141	14	do.	.61		24		{ 24 155 } 48 212		24	6.1	3,360	3,800	3,020	3,670	121			
147	14	do.	.62		24	27			25	6.3	4,560	4,890	4,560	5,220	114	6,410		
148	14	do.	.62		72				25	7.0	5,020	4,810	5,310	5,960	112	6,300		
149	14	do.	.62		24		48	100	25	6.5	5,370	5,510	5,240	5,450	104	5,300		
150	14	do.	.62		24		48	155	25	6.2	3,360	3,870		3,400		4060(120)		
151	14	do.	.62		24		48	212	25	6.1	3,300	3,750	2,840	3,650	129	4,110		
152	14	do.	.62		24	20			344	6.3	4,570	5,040	5,100	5,040	99			
153	14	do.	.62		24	20			56	6.5	4,830	4,910	5,120	6,280	123			
154	14	do.	.62		24	20			28	6.5	4,760	4,850	5,450	5,620	103			
155	14	do.	.62		24	20			14	6.5	4,250	4,960	4,570	5,770	126			
156	14	do.	.62		24	20				6.3	4,710	4,860	4,380	4,960	113			
157	14	One-half Portland A, one-half Portland B.	.74	40 per cent blast-furnace slag admixed.	24	20				6.5	2,420	3,960	6,030	4,230	70	2,340		
158	18	do.	.63	10 per cent blast-furnace slag admixed.	24	20			35	6.0	2,880	4,180	5,440	5,170	95	3,930		
159	18	do.	.74	40 per cent blast-furnace slag admixed.	24	20			35	6.7	2,410	4,190	5,670	5,560	98	5,220		
160	18	do.	.63	10 per cent blast-furnace slag admixed.	24		48	155	35	6.4	3,850	4,010	5,300	4,800	91	3,540		
161	18	do.	.74	40 per cent blast-furnace slag admixed.	24		48	155	35	6.9	3,800	4,020	5,450	4,830	89	5,640		
162	18	do.	.63	4 per cent calcium chloride admixed.	24	20				5.7	2,870	4,330	5,530	3,910	71	1,710		
163	18	do.	.70	8 per cent calcium chloride admixed.	21	20			35	7.1	2,200	3,010	4,280	3,550	83	3,130		
164	18	do.	.63	4 per cent calcium chloride admixed.	24	20			35	5.9	2,940	4,000	5,450	5,050	93	5,240		
165	18	do.	.70	8 per cent calcium chloride admixed.	24		48	155	35	7.1	3,800	3,640	4,600	3,300	72	3,860		

¹ Figures in parentheses show percentage of normal strength as indicated by parallel tests on cylinders from same batches stored in tap water in the laboratory.

TABLE 2.—Description and tests of all laboratory standard concrete cylinders immersed in Medicine Lake, S. Dak., and parallel tests on specimens stored in laboratory tanks—Continued

Series No.	Cement laboratory No.	Cement	Water ratio	Admixture, impregnation or surface treatment	Curing method						Average of compression tests						
					Time in moist closet	Time in water	Time in water, vapor or steam	Temperature of water or steam	Time in air	Absorption at 21 days	Tank specimens				Lake specimens		
											7 days	28 days	1 year	1 year	Percentage of normal strength as indicated by tank specimens at one year	2 years	3 years
					Hours	Days	Hours	°F.	Days	Per cent	Lbs. per sq. in.	Lbs. per sq. in.	Lbs. per sq. in.	Lbs. per sq. in.		Lbs. per sq. in.	Lbs. per sq. in.
166.....	18	One-half Portland A, one-half Portland B.	0.63	4 per cent calcium chloride admixed.	24			155	35	5.8	4,810	4,920	4,990	4,460			
167.....	18	do.	.59	4 per cent Cal admixed.	24	20				5.7	3,200	4,150	4,860	3,910	80	1,380	
168.....	18	do.	.60	8 per cent Cal admixed.	21	20			35	5.8	3,180	4,290	5,570	4,620	83	3,690	
169.....	18	do.	.59	4 per cent Cal admixed.	24	20			35	5.8	3,250	4,280	4,550	4,800	105	3,620	
170.....	18	do.	.60	8 per cent Cal admixed.	24		48	155	35	6.0	4,990	4,670	5,030	5,010	100	6,200	
171.....	18	do.	.59	4 per cent Cal admixed.	24		48	155	35	6.0	4,550	4,690	5,140	4,980	97	5,560	
172.....	18	do.	.59	do.	24	20			35	5.6	2,870	4,190	5,300	4,370	82	2,650	
173.....	18	do.	.61	20 per cent ironite admixed.	24	20			35	5.9	2,950	3,980	4,890	5,020	102	4,000	
174.....	18	do.	.59	do.	24		48	155	35	6.1	4,020	4,390	4,330	3,910	90	2,670	
175.....	18	do.	.61	20 per cent Ironite admixed.	24		48	155	35	6.1	3,880	4,440	5,510	4,630	84	4,830	
176.....	18	do.	.61	do.	24		48	100	35	6.1	3,680	4,220	5,250	5,190	99	5,210	
177.....	18	do.	.59	do.	24	20			35	5.9	2,520	4,200	4,930	3,670	74	1,910	
178.....	18	do.	.74	20 per cent volcanic ash admixed.	24	20			35	6.8	1,900	3,210	5,450	2,960	54	1,970	
179.....	18	do.	.59	do.	24		48	155	35	5.8	4,110	4,510	5,210	3,820	73	2,380	
180.....	18	do.	.74	20 per cent volcanic ash admixed.	24		48	155	35	7.6	3,240	3,340	4,440	3,600	81	4,510	
181.....	18	do.	.74	do.	24		48	100	35	7.3	2,710	2,820	5,130	4,290	84	3,590	
182.....	18	do.	.59	do.	24	20			35	6.0	2,800	4,010	5,780	4,450	77	2,100	
183.....	18	do.	.63	3 per cent Alkagel "A" admixed.	24	20			35	5.3	2,260	3,560	4,570	3,520	77	2,140	
184.....	18	do.	.59	do.	24		48	155	35	6.0	4,070	4,530	5,100	4,240	83	4,700	
185.....	18	do.	.63	3 per cent Alkagel "A" admixed.	24		48	155	35	5.0	3,280	3,940	4,570	3,300	72	3,900	
186.....	18	do.	.63	do.	24		48	100	35	5.4	3,120	3,730	3,820	3,550	93	3,060	
208-212.....	25	High alumina B.	.51		24	20			35	4.9	6,700	6,780	6,260	7,420	118	7,690(95)	7,240(91)
243-247.....	28	Special A 1.	.60		24	20			35	6.2	2,910	3,770	5,370	4,030	75	780(12)	0
252-253.....	58	High alumina C.	.53		24	20			35	5.4	7,730	7,250	7,480	7,010	94	7,430	
256.....	61	One-half Portland A, one-half Portland B.	.59		24	20			35	5.8	2,780	3,950	5,200	1,480	28	120	
257.....	63	Portland A.	.59		24	20			35	5.8	2,720	4,290	5,490	3,190	58	690	
258.....	62	Portland B1.	.59		24	20			35	5.8	2,960	3,790	4,580	800	17	0	
259.....	33	Portland D.	.59		24	20			35	5.4	3,340	4,780	5,900	1,590	27	0	
260.....	34	Portland C.	.66		24	20			35	6.1	3,340	4,540	5,980	5,090	85	4,390	
286.....	27	High alumina B.	.44		24	20			35	5.1	7,490	6,820	8,080	8,460	106		
287-288.....	27	do.	.53		24	20			35	4.9	7,080	6,620	8,160	8,210	101	7,400	
289.....	27	do.	.67		24	20			35	5.3	6,560	5,680	6,290	6,160	98		
290.....	27	do.	.81		24	20			35	5.8	4,640	4,840	4,150	5,080	122		
291.....	58	High alumina C.	.79		24	20			35	6.5	4,360	4,630	2,910	4,370	150		
296.....	61	One-half Portland A, one-half Portland B.	.62		24		48	212	53	6.5	3,330	3,660	5,020	4,390	88		
297.....	63	Portland A.	.60		24		48	212	53	6.4	3,650	3,660	4,680	4,600	98		
298.....	62	Portland B1.	.62		24		48	212	53	6.9	3,700	3,830	4,580	4,480	98		
299.....	33	Portland D.	.60		24		48	212	53	6.4	3,690	3,770	4,620	4,270	92		
300.....	34	Portland C.	.67		24		48	212	53	6.6	3,880	4,540	4,940	4,890	99		
301.....	61	One-half Portland A, one-half Portland B.	.62		24		48	212	53	6.5	3,690	3,610	4,500	4,480	100		
302.....	35	Portland E.	.60		24		48	212	53	6.2	3,540	3,530	4,550	4,450	98		
303.....	37	Portland F.	.60		24		48	212	53	6.0	3,550	3,900	5,060	4,920	97		
304.....	39	Portland G1.	.60		24		48	212	53	6.2	3,740	3,920	4,990	4,680	94		
305.....	41	Portland H.	.60		24		48	212	53	6.1	3,730	3,840	4,860	4,410	91		
306.....	61	One-half Portland A, one-half Portland B.	.62		24		48	212	53	6.2	3,280	3,920	4,480	4,410	98		
307.....	40	Portland I.	.60		24		48	212	53	5.9	3,420	4,150	5,090	4,190	82		
308.....	42	Portland J.	.62		24		48	212	53	6.4	2,630	3,240	4,680	3,480	74		
309.....	55	Portland K1.	.62		24		48	212	53	6.5	3,200	4,200	4,950	4,910	99		
310.....	60	Portland L1.	.60		24		48	212	53	6.2	3,260	3,870	5,060	4,460	88		
311.....	65	Portland M.	.60		24		48	212	53	5.5	4,660	5,580	5,810	5,570	96		
313.....	65	do.	.60		24	20			35	3.9	3,150	3,950	6,050	2,250	37		
314.....	65	do.	.60		24		48	155	53	6.0	4,460	4,010	6,170	5,430	88		
315.....	65	do.	.60		24		48	100	53	6.2	3,740	3,800	5,970	4,230	71		
351.....	74	One-half Portland A, one-half Portland B.	.62		24		12	190	54	6.6	2,650	2,940	5,320	1,730	33		
352.....	74	do.	.62		24		12	212	54	6.9	2,450	2,910	5,230	4,240	81		
353.....	74	do.	.62		24		12	235	54	7.3	1,900	2,750	4,780	3,800	79		
354.....	74	do.	.62		24		12	260	54	7.1	2,270	2,490	4,880	3,760	77		
355.....	74	do.	.62		24		12	285	54	7.2	2,260	2,870	4,340	3,870	89		

1 Mix 1:2.

3 Mix 1:4.

4 Mix 1:5.

TABLE 2.—Description and tests of all laboratory standard concrete cylinders immersed in Medicine Lake, S. Dak., and parallel tests on specimens stored in laboratory tanks—Continued

Series No.	Cement laboratory No.	Cement	Water ratio	Admixture, impregnation or surface treatment	Curing method					Absorption at 21 days	Average of compression tests						
					Time in moist closet	Time in water	Time in water, vapor or steam	Temperature of water or steam	Time in air		Tank specimens				Lake specimens		
											7 days	28 days	1 year	1 year	Percentage of normal strength as indicated by tank specimens at one year	2 years	3 years
					Hours	Days	Hours	° F.	Days	Per cent	Lbs. per sq. in.	Lbs. per sq. in.	Lbs. per sq. in.	Lbs. per sq. in.	Lbs. per sq. in.	Lbs. per sq. in.	Lbs. per sq. in.
361.....	80	Portland F.....	0.62	-----	24	20	-----	-----	35	6.3	2,870	4,320	6,350	0	0	-----	-----
362.....	79	Portland N1.....	.62	-----	24	20	-----	-----	35	6.1	3,220	4,600	6,970	4,000	57	-----	-----
363.....	89	Portland O1.....	.62	-----	24	20	-----	-----	35	5.9	4,010	5,220	6,730	670	10	-----	-----
364.....	81	Portland E.....	.62	-----	24	20	-----	-----	35	5.8	3,340	4,770	7,320	2,910	40	-----	-----
365.....	83	Portland H.....	.62	-----	24	20	-----	-----	35	6.1	3,940	4,770	6,280	3,680	59	-----	-----
366.....	77, 96	Portland G2.....	.64	-----	24	20	-----	-----	35	6.0	4,050	4,820	5,900	1,930	33	-----	-----
367.....	82, 95	Portland C.....	.67	-----	24	20	-----	-----	35	6.8	3,220	4,320	6,270	4,790	76	-----	-----
368.....	85, 94	Portland D.....	.64	-----	24	20	-----	-----	35	6.4	3,390	4,130	5,730	2,980	52	-----	-----
369.....	86, 99	Portland I.....	.64	-----	24	20	-----	-----	35	6.1	3,590	4,370	6,110	4,470	73	-----	-----
370.....	87, 88	Portland P.....	.64	-----	24	20	-----	-----	35	6.1	3,730	5,040	6,540	810	12	-----	-----
371.....	84	Portland L2.....	.64	-----	24	20	-----	-----	35	5.8	3,690	5,000	5,770	3,170	55	-----	-----
372.....	97	Portland K2.....	.64	-----	24	20	-----	-----	35	5.4	3,120	4,440	5,720	4,600	80	-----	-----
373.....	98	Portland K1.....	.64	-----	24	20	-----	-----	35	6.1	4,220	4,970	7,050	3,400	48	-----	-----
374.....	74	One-half Portland A, one-half Portland B.....	.62	Two coats Inertol.....	24	20	-----	-----	35	6.7	2,930	4,870	5,570	5,390	97	-----	-----
391.....	74	One-half Portland A, one-half Portland B.....	.62	-----	24	20	-----	-----	35	6.3	3,130	4,890	6,060	2,480	41	-----	-----
392.....	74	do.....	.76	33 per cent Trass admixed.	24	20	-----	-----	35	7.3	2,710	3,830	5,790	5,040	87	-----	-----
392a ³	74	do.....	.85	do.....	24	20	-----	-----	35	7.1	2,430	3,450	5,260	4,170	79	-----	-----
393.....	74	do.....	.93	66 per cent Trass admixed.	24	20	-----	-----	35	9.1	1,930	3,350	4,600	4,400	96	-----	-----
393a ⁴	74	do.....	1.09	do.....	24	20	-----	-----	35	8.4	1,440	2,670	4,270	3,660	86	-----	-----
394.....	86	Portland I.....	.78	33 per cent Trass admixed.	24	20	-----	-----	35	6.8	3,330	4,690	6,620	5,300	80	-----	-----
394a ³	86	do.....	.87	do.....	24	20	-----	-----	35	6.7	3,000	3,850	6,170	4,140	67	-----	-----
395.....	86	do.....	.95	66 per cent Trass admixed.	24	20	-----	-----	35	7.6	2,710	3,770	6,060	4,300	71	-----	-----
395a ⁴	86	do.....	1.09	do.....	24	20	-----	-----	35	7.3	2,280	3,380	4,090	3,810	93	-----	-----
396.....	70, 71	High alumina B.....	.44	-----	24	20	-----	-----	35	5.6	5,300	5,760	6,110	5,780	95	-----	-----
397.....	70, 71	do.....	.53	-----	24	20	-----	-----	35	4.9	6,660	7,120	8,100	7,580	94	-----	-----
398.....	70, 71	do.....	.59	-----	24	20	-----	-----	35	5.3	6,890	7,210	8,630	7,390	86	-----	-----
399.....	70, 71	do.....	.73	-----	24	20	-----	-----	35	6.3	5,950	5,710	6,440	5,400	84	-----	-----
400.....	70, 71	do.....	.88	-----	24	20	-----	-----	35	7.6	5,140	4,230	3,480	3,790	109	-----	-----
401.....	100	Portland Q1.....	.62	-----	24	20	-----	-----	35	6.4	2,900	4,310	5,110	0	0	-----	-----
402.....	101	Portland R.....	.62	-----	24	20	-----	-----	35	6.3	3,680	4,940	6,290	2,440	39	-----	-----
403.....	102	Portland N2.....	.62	-----	24	20	-----	-----	35	5.9	3,860	5,040	6,630	4,410	67	-----	-----
404.....	103	Portland S.....	.62	-----	24	20	-----	-----	35	6.0	3,010	4,380	5,470	3,160	58	-----	-----
405.....	104	Portland T.....	.62	-----	24	20	-----	-----	35	6.4	2,940	4,380	5,530	280	5	-----	-----
412.....	105	Portland Q2.....	.66	2½ per cent Celite admixed.	24	20	-----	-----	35	6.4	3,530	5,050	6,670	3,760	56	-----	-----
413.....	105	do.....	.62	-----	24	20	-----	-----	35	5.7	3,520	5,390	6,500	4,300	66	-----	-----
414.....	106	Portland U.....	.62	-----	24	20	-----	-----	35	6.1	3,030	4,410	6,430	2,490	39	-----	-----
415.....	107	Portland V.....	.62	-----	24	20	-----	-----	35	6.5	2,830	4,540	5,920	2,940	50	-----	-----
416.....	108	Portland W.....	.62	-----	24	20	-----	-----	35	5.8	3,670	5,010	6,510	1,490	23	-----	-----
421 ⁵	74	One-half Portland A, one-half Portland B.....	.37	-----	24	20	-----	-----	35	15.8	6,330	7,630	8,600	8,990	105	-----	-----
422 ⁵	97, 98	Portland K1 and K2.....	.35	-----	24	20	-----	-----	35	10.5	6,890	9,880	9,560	10,230	107	-----	-----
423 ⁵	86, 99	Portland I.....	.38	-----	24	20	-----	-----	35	12.1	8,020	9,830	9,720	9,380	97	-----	-----
424 ⁵	82, 95	Portland C.....	.42	-----	24	20	-----	-----	35	14.8	7,080	8,560	10,120	8,920	88	-----	-----
425 ⁵	83	Portland H.....	.36	-----	24	20	-----	-----	35	13.6	6,140	7,290	10,480	9,910	95	-----	-----
429 ⁵	70, 71	High alumina B.....	.35	-----	24	20	-----	-----	35	9.7	9,460	8,420	10,440	9,330	89	-----	-----
430.....	97, 98	Portland K1 and K2.....	.64	-----	24	-----	12	285	54	6.6	2,960	3,470	3,950	3,680	93	-----	-----
431.....	86, 99	Portland I.....	.64	-----	24	-----	12	285	54	6.1	3,650	4,150	4,470	4,290	96	-----	-----
432.....	82, 95	Portland C.....	.67	-----	24	-----	12	285	54 ⁴	6.9	3,330	3,820	5,530	4,930	89	-----	-----
433.....	83	Portland H.....	.64	-----	24	-----	12	285	54	6.5	3,460	3,950	5,110	4,840	95	-----	-----
434.....	74	One-half Portland A, one-half Portland B.....	.66	-----	24	20	-----	-----	35	6.4	2,590	4,030	5,050	4,870	96	-----	-----
435.....	70, 71, 74	5 per cent high alumina B, 47.5 per cent Portland A, and 47.5 per cent Portland B.....	.66	-----	24	20	-----	-----	35	6.3	2,630	3,880	5,490	4,100	75	-----	-----
436.....	70, 71, 74	10 per cent high alumina B, 45 per cent Portland A and 45 per cent Portland B.....	.66	-----	24	20	-----	-----	35	6.2	2,260	3,760	4,820	2,720	56	-----	-----
437.....	70, 71, 74	20 per cent high alumina B, 40 per cent Portland A and 40 per cent Portland B.....	.66	-----	24	20	-----	-----	35	7.0	1,360	2,830	3,640	3,320	91	-----	-----

³ Mix 1:4⁴ Mix 1:5.⁵ Neat cement.

TABLE 2.—Description and tests of all laboratory standard concrete cylinders immersed in Medicine Lake, S. Dak., and parallel tests on specimens stored in laboratory tanks—Continued

Series No.	Cement laboratory No.	Cement	Water ratio	Admixture, impregnation or surface treatment	Curing method							Average of compression tests						
					Time in moist closet	Time in water	Time in water, vapor or steam	Temperature of water or steam	Time in air	Absorption at 21 days	Tank specimens			Lake specimens				
											7 days	28 days	1 year	1 year	Percentage of normal strength as indicated by tank specimens at one year	2 years	3 years	
					Hours	Days	Hours	° F.	Days	Per cent	Lbs. per sq. in.	Lbs. per sq. in.	Lbs. per sq. in.	Lbs. per sq. in.	Lbs. per sq. in.	Lbs. per sq. in.	Lbs. per sq. in.	
438.....	70, 71	High alumina B	0.66		24	20			35	5.8	7,080	7,790	7,820	7,220	92			
464.....	109	Portland B2.....	.64		24	20			35	5.8	3,740	5,010	7,070	5,950	84			
465.....	110	Portland B2.....	.64		24	20			35	5.5	3,660	4,920	6,910	4,850	70			
466.....	111	Portland B3.....	.64		24	20			35	5.7	3,810	5,020	5,860	3,960	68			
467 ^a	112	Portland X.....	.54		24	20			35		2,830	4,390	4,820	4,830	100			
468.....	113	Portland O3.....	.64		24	20			35	6.3	2,140	3,940	6,000	5,590	93			
549.....	129	One-half Portland A, one-half Portland B.	.62		24	20	Water at 212°-½ min.	{	3									
									32	5.8	3,790	4,770						
550.....	129	do.....	.62	One coat boiled linseed oil at 70° F.	24	20			35	5.8	3,540	4,960						
551.....	129	do.....	.62	Two coats boiled linseed oil at 70° F.	24	20			35	5.8	3,730	5,260						
552.....	129	do.....	.62	One coat boiled linseed oil at 225° F.	24	20			35	5.5	3,430	4,680						
553.....	129	do.....	.62	Two coats boiled linseed oil at 225° F.	24	20			35	5.8	3,750	4,960						
554.....	129	do.....	.64		24	20			35	6.3	3,570	4,760						
555.....	128	Portland Y.....	.64		24	20			35	6.0	3,780	4,670						
556.....	127	Special A2.....	.64		24	20			35	6.0	3,200	4,720						
557.....	128	Portland Y.....	.73		24	20			35	6.4	2,760	4,250						
558.....	127	Special A2.....	.73		24	20			35	6.6	2,280	4,480						
559 ^a	129	One-half Portland A, one-half Portland B.	.62		24	20 days in damp sand.			35	8.3	3,340	4,530						
560 ^a	128	Portland Y.....	.62		24	20 days in damp sand.			35	8.2	3,550	5,160						
561 ^a	127	Special A2.....	.62		24	20 days in damp sand.			35	7.9	3,260	4,720						
562 ^a	128	Portland Y.....	.71		24	20 days in damp sand			35	9.2	2,720	4,290						
563 ^a	127	Special A2.....	.71		24	20 days in damp sand			35	9.1	2,290	4,520						
564-565.....	129	One-half Portland A, one-half Portland B.	.62	2½ per cent powdered fuel ash admixed.	24	20			35	6.3	3,580	5,100						
566.....	129	do.....	.64	5 per cent powdered fuel ash admixed.	24	20			35	6.4	3,300	4,760						
567-568.....	129	do.....	.67	10 per cent powdered fuel ash admixed.	24	20			35	6.3	3,390	5,080						
573.....	130	Special B.....	.62		24	20			35	6.3	4,140	4,800						
574.....	131	Special C.....	.62		24	20			35	6.6	3,490	4,560						
575.....	132	Portland Z.....	.62		24	20			35	6.0	4,760	5,160						
576.....	133	Portland A A.....	.62		24	20			35	6.0	3,460	4,820						
577.....	134	Portland G1.....	.62		24	20			35	6.3	3,910	5,610						
578.....	129	One-half Portland A, one-half Portland B.	.62	1 per cent Truscon admixed.	24	20			35		3,370	4,700						
579.....	129	do.....	.62	2 per cent Truscon admixed.	24	20			35		3,370	4,580						
580.....	129	do.....	.62	4 per cent Truscon admixed.	24	20			35		3,020	3,980						
581.....	129	do.....	.62	6 per cent barium chloride admixed.	24	20			35	6.5	3,530	4,570						
582.....	129	do.....	.62	12 per cent barium chloride admixed.	24	20			35	6.7	3,240	4,460						
583.....	139	do.....	.62		24	20			35	6.3	3,810	4,950						
584.....	139	do.....	.62		24	20	1 min.	212	35	6.3	3,740	5,150						
585.....	139	do.....	.62		24	20	1 min.	212	55	6.4	2,770	2,860						
586-587.....	139	do.....	.62		24	20	1 min.		54	6.6	2,970	2,920						
634.....	139	do.....	.64	2 per cent Celite admixed.	24	20			35	6.1	3,260	5,340						
635-636.....	139	do.....	.64		24		48	212	53	6.3	3,960	4,330						
637-638.....	139	do.....	.64	2 per cent Celite admixed.	24		48	212	53	6.2	4,160	4,730						
639.....	139	do.....	.64	2 per cent Colloy admixed.	24	20			35	6.5	3,630	4,610						
640.....	139	do.....	.64		24	20			35	6.5	3,320	4,470						
641.....	139	do.....	.64		24		48	212	53	6.9	3,590	3,760						
642-643.....	139	do.....	.64	2 per cent Colloy admixed.	24		48	212	53	6.8	3,870	4,100						

^a Mix 1 : 2.^b Neat cement.^c Mix 1 : 2.25 and fineness modulus 3.10.

TABLE 3.—Description and tests of standard Ottawa sand cylinders immersed in Medicine Lake, N. Dak., and parallel tests on specimens stored in laboratory tanks

(Unless otherwise noted the mix is 1:3. Each test result, with a few exceptions, is an average of five cylinders made on different days.)

Series No.	Cement laboratory No.	Cement	Water ratio	Admixture, impregnation or surface treatment	Curing method					Absorption at 21 days	Average of compression tests						
					Time in moist closet	Time in water	Time in water, vapor or steam	Temperature of water, vapor, or steam	Time in air		Tank specimens			Lake specimens			
											7 days	28 days	1 year	1 year	Percentage of normal strength as indicated by tank specimens at one year	2 years ¹	3 years ¹
Per cent.	Lbs. per sq. in.	Lbs. per sq. in.	Lbs. per sq. in.	Lbs. per sq. in.	Lbs. per sq. in.	Lbs. per sq. in.	Lbs. per sq. in.										
107-----	17	One-half Portland A, one-half Portland B.	0.64		Hours 24	Days	Hours 48	°F. 155	Days 25	9.8	2,660	3,080	3,580	770	23	0	0
108-----	17	do	.64		24		{ 24 155 } 24 212		25	9.8	2,590	2,910	2,930	2,540	87		3,360
109-----	17	do	.64		24		{ 72 155 } 24 212		24	9.6	2,670	3,220	3,490	930	27	0	0
110-----	17	do	.64		24		{ 48 155 } 24 212		24	9.5	2,600	3,160	2,940	2,720	93		3,160
111-----	17	do	.64		24		{ 24 155 } 48 212		24	9.7	2,510	31,30	2,630	2,450	93		2,820
112-----	17	do	.64		24	27				10.0	1,650	2,630	3,540	1,790	51	0	0
113-----	17	do	.64		72				25	10.7	2,000	2,000	2,980	840	28	0	0
114-----	17	do	.64		24		48	100	25	10.1	2,270	2,790	3,040	630	21	0	0
115-----	17	do	.64		24		48	155	25	10.1	2,730	2,900	3,400	860	25	0	0
116-----	17	do	.64		24		48	212	25	10.3	2,220	2,470	2,500	2,430	97	2,580	
127-----	17	do	.64		24		48	155	53	9.1	2,530	2,710	3,460	2,280	66		
128-----	17	do	.64		24		{ 24 155 } 24 212		53	9.1	2,350	2,480	2,830	3,180	112		
129-----	17	do	.64		24		{ 72 155 } 48 212		52	9.0	2,700	3,120	3,320	1,970	59		
130-----	17	do	.64		24		{ 48 155 } 24 212		52	9.1	2,750	2,720	2,830	2,080	109		
131-----	17	do	.64		24		{ 24 155 } 48 212		52	9.2	2,630	2,470	2,600	2,560	98		
132-----	14	High alumina A.	.65		24		48	155	25	8.1	2,250	2,910	2,760	2,790	101	2,930	
133-----	14	do	.65		24		{ 24 155 } 24 212		25	8.0	2,280	2,680	2,080	2,900	139	3,140	
134-----	14	do	.65		24		72	155	24	8.0	2,470	2,850	2,440	2,840	116	3,120	
135-----	14	do	.65		24		{ 48 155 } 24 212		24	7.8	2,620	2,960	2,650	2,590	98	2,930	
136-----	14	do	.65		24		{ 24 155 } 48 212		24	7.8	2,550	2,930	2,630	2,580	98	3,170	
142-----	14	do	.63		24	27				8.7	2,910	3,600	3,590	4,060	113		
143-----	14	do	.63		72				25	9.3	4,220	4,430	4,220	4,310	102		
144-----	14	do	.63		24		48	100	25	9.1	4,320	4,640	3,750	4,490	120		
145-----	14	do	.63		24		48	155	25	8.1	2,200	2,760	2,780	2,620	94		
146-----	14	do	.63		24		48	212	25	7.8	2,500	2,870	2,500	3,010	120		
248-250.	10	Special A3.	.63		24	20			35	9.5	1,430	2,730	3,750	3,280	87	1,930(45)	800(20)
254-255.	58	High alumina C.	.60		24	20			35	9.3	5,010	4,850	2,920	4,340	149	4,380	
276 ²	27	High alumina B.	.44		24	20			35	6.3	6,210	7,680	6,490	8,000	123		
277-278.	27	do	.39		24	20			35	8.3	5,060	4,830	5,110	5,560	109	5,060	
279 ³	27	do	.73		24	20			35	10.2	3,380	3,090	3,250	3,730	115		
280 ⁴	27	do	.99		24	20			35	11.8	2,160	1,920	1,680	700	42	0	
292 ⁴	58	High alumina C.	.90		24	20			35	12.3	2,130	1,820	1,130	920	81	0	
293-----	19	One-half Portland A, one-half Portland B.	.64		24	20			35	9.9	1,420	2,240	3,850	2,140	56	1,340	
294-----	19	do	.64	Sulphur impregnated.	24	20			35	9.9	1,350	2,210	3,130	1,500	48	970	
295-----	19	do	.67	10 per cent sulphur admixed.	24	20			35	9.7	1,150	1,250	1,870	0	0	0	
375-----	74	do	.64	Two coats Inertol at 70° F.	24	20			35	10.5	1,430	2,790	3,640	2,970	82		

¹ Figures in parentheses show percentage of normal strength as indicated by parallel tests on cylinders from same batches stored in tap water in the laboratory.² Mix 1:2.³ Mix 1:4.⁴ Mix 1:5.

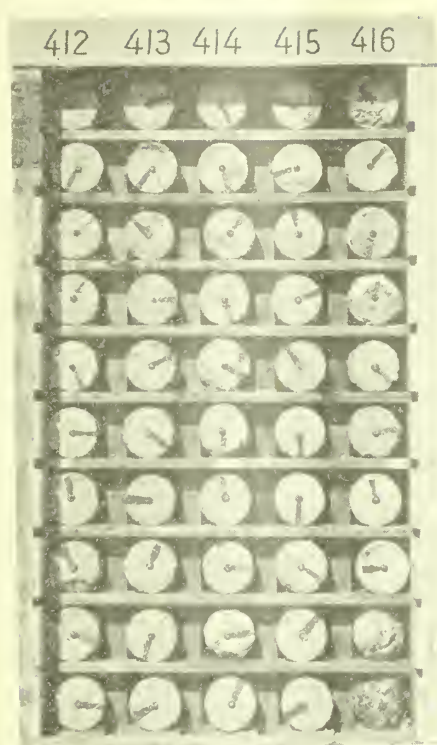


FIG. 8.—CYLINDERS MADE WITH DIFFERENT BRANDS OF STANDARD PORTLAND CEMENT AFTER ONE YEAR IN LAKE

ANALYSIS OF CONCRETE ARCHES

"Analysis of concrete arches," by W. P. Linton and C. D. Geisler, which appeared in the June and July, 1927, issues of *Public Roads*, is now available as a separate publication. The authors have developed a method which greatly reduces the tedious and complicated processes usually followed in the calculation of arch stresses. The method makes use of a set of standard forms, based on formulas not greatly different from those found in other texts. Many of the entries on the forms are the same for all arches and can be printed on them; and the authors have presented the various steps in filling out the forms in such a way that the work is largely mechanical and the chances of error greatly reduced.

A copy may be secured by addressing the United States Department of Agriculture, Office of Information, Washington, D. C.

REGIONAL HIGHWAY PLANNING SURVEY FOR CLEVELAND

Field work has been under way since September 3 upon the Cleveland regional highway planning survey, embracing a territory within a radius of approximately 30 miles of Cleveland, Ohio. The survey is being carried on by the United States Bureau of Public Roads and Cuyahoga County as the principal co-operators. Information and assistance is also being rendered by the several administrative organizations responsible for highway development in the Cleveland region.

The preliminary work included a comprehensive study of traffic density—its distribution, origin, and destination—on the various routes in the area. This is being followed by a study of the traffic capacity of the various county highways and the principal entrances into the city of Cleveland and an analysis of the present highway system. Recommendations as to the location of new highways or relocation of present routes will involve consideration of the influence of such factors as topography, waterways, railroad terminals and yards, large industrial plants, suburban developments, and special use areas. With this information there is to be developed a plan of highway improvement adequate for anticipated traffic and designed especially to eliminate highway congestion. An improvement budget for a period sufficient to carry the complete plan into effect will also be prepared.

At the present time collection of field data is practically completed and a detailed analysis of this material is in progress. Traffic records were taken at 263 points in the area and show the daily and hourly volume of passenger cars, motor trucks, and motor busses on all important highways. Information was also taken as to weights of motor trucks. Records as to the origin and destination were taken for approximately 100,000 vehicles.

The "roughometer" described in the issue of *Public Roads* for September, 1926, was used in making a condition survey of the improved roads in the area. This instrument is attached to a passenger car and gives an accumulated record of the roughness of the surfaces over which the vehicle is driven.

Another new instrument, used for the first time in this survey, gives a record of the speed at which traffic moves at all points along a highway. This instrument is being used in studying traffic congestion and traffic capacity of various widths of highway.

GASOLINE TAXES FOR FIRST SIX MONTHS OF 1927

State	Gross tax assessed prior to deduction of refunds	Exemption refunds (deducted from gross tax)	Total tax earnings on fuel for motor vehicles ¹	Disposition of total gasoline tax earnings ¹				Tax rates, first six months of 1927		Net gallons of gasoline taxed and used by motor vehicles	Estimated additional gallons (not taxed) used by motor vehicles		
				Collection costs (half year)	Construction and maintenance of rural roads		State and county road-bond payments	Miscellaneous	Cents per gallon			Date of rate change	
					State highways	Local roads			Jan. 1				June 30
Alabama	\$2,653,637		\$2,653,637	\$10,302	\$1,300,778	\$1,342,557		2	4	Jan. 25	68,096,599		
Arizona	662,729	\$144,130	518,599		259,300	259,299		3	3		17,286,641		
Arkansas	1,741,086	180,527	1,560,559	46,817	358,929	171,661	³ \$983,152	4	5	June 9	39,013,981		
California	9,677,342	641,408	9,035,934	19,918	4,508,008	4,508,008		2	2	(⁴)	451,796,681		
Colorado	1,237,359	43,282	1,194,077	(⁵)	730,359	463,718		2	3	May 1	57,701,371		
Connecticut	1,307,024		1,307,024		1,307,024			2	2		65,351,209		
Delaware	273,203	6,112	267,091		267,091			2	3	Mar. 24	10,329,754		
Florida	5,355,219		5,355,217	2,100	4,014,838	1,338,279		4	4	(⁶)	133,880,422		
Georgia	3,071,528		3,071,528	2,100	2,192,448	876,980		3 1/2	3 1/2		87,757,935		
Idaho	637,373	30,679	606,694	5,203	601,491			3	4	Feb. 28	15,750,489		
Illinois								0	7	No. tax.		256,900,000	
Indiana	4,697,106	132,395	4,564,711	7,398	3,038,209	1,519,104		3	3		152,157,038		
Iowa	2,865,439	88,256	2,777,183	4,956	924,076	1,848,151		2	2	(⁸)	138,859,170		
Kansas	2,217,334	326,089	1,891,245	(⁹)	1,640,500	250,745		2	2		94,562,249		
Kentucky	2,472,015		2,472,015		2,472,015			5	5		49,348,018		
Louisiana	1,411,554		1,411,554	(¹⁰)	1,411,554			2	2		70,577,705		
Maine	761,073	18,148	742,925	5,341	368,792	368,792		3	3		24,461,721		
Maryland	1,831,099	57,257	1,774,442	1,250	1,418,554		¹² \$354,638	2	4	Apr. 1	57,069,042		
Massachusetts								0	0	No. tax.		148,200,000	
Michigan	5,414,065	380,267	5,033,798	14,718	2,994,725	1,024,355	¹³ 1,000,000	2	2	(¹⁴)	251,689,874		
Minnesota	2,367,545	93,218	2,274,327	(¹⁵)	2,274,327			2	2		115,891,613		
Mississippi	2,136,296		2,136,296	1,800	1,003,006	1,068,244	¹⁶ 63,246	4	4		51,826,257		
Missouri	2,968,788	47,723	2,921,065	24,017	2,897,048			2	2		146,053,264		
Montana	443,439	15,076	428,363	3,150	425,213			3	3	Jan. 1	14,278,756		
Nebraska	1,477,472	7,408	1,470,064	3,639	1,466,425			2	2		73,503,208		
Nevada	217,604	18,107	199,497		99,749	99,748		4	4		4,987,419		
New Hampshire	421,366	7,765	413,581		413,581			2	3	May 1	18,310,415		
New Jersey								0	17	No. tax.		137,500,000	
New Mexico	¹⁸ 614,200		¹⁸ 614,200	11,590	602,610			3	5	Mar. 7	14,081,758		
New York								0	0	No. tax.		375,100,000	
North Carolina	4,108,931	176,387	3,932,544	8,218	1,705,745		¹⁹ 2,218,581	4	4		98,313,590		
North Dakota	467,210	71,277	395,933		395,933			2	2		19,796,669		
Ohio	7,722,220	182,394	7,539,826		4,739,600	1,272,830	³⁰ 1,527,396	2	3	May 25	336,902,146		
Oklahoma	3,208,634	12,258	3,196,376		2,130,919	1,065,457		3	3		110,262,328		
Oregon	1,690,893	77,684	1,613,209	4,149	1,609,060			3	3		54,081,297		
Pennsylvania	6,101,696		6,101,696	(²¹)	4,067,797	2,033,899		2	2	(²²)	305,084,825		
Rhode Island	326,848	25,877	300,971	(²³)	253,953		47,018	1	2	June 1	25,395,311		
South Carolina	2,214,215	3,579	2,210,636	(²⁴)	1,360,794	849,842		5	5		44,212,717		
South Dakota	1,095,630	135,000	960,630	2,783	957,847			3	3	(²⁵)	32,616,933		
Tennessee	1,983,139		1,983,139	19,831	1,963,308			3	3		66,104,639		
Texas	5,975,553		5,975,553		4,481,665			1	3	Mar. 16	271,417,466		
Utah	567,794		567,794	5,375	²⁷ 212,419		²⁸ 350,000	3 1/2	3 1/2		16,222,709		
Vermont	275,536		275,536		275,536			2	3	June 1	12,130,292		
Virginia	3,298,685	176,167	3,122,518	(²⁹)	2,081,679	1,040,839		4 1/2	4 1/2		69,259,098		
Washington	1,734,604	103,378	1,631,226		1,631,226			2	2		81,561,285		
West Virginia	1,458,314	42,257	1,416,057	(³⁰)	1,416,057		(³⁰)	3 1/2	3 1/2	(³¹)	40,458,768		
Wisconsin	2,618,397	92,339	2,526,058	(³²)	1,085,000	1,441,058		2	2		126,302,901		
Wyoming	257,275	367	256,908	³³ 8	256,900			2 1/2	3	Mar. 1	9,244,481		
District of Columbia	551,405	2,800	548,605					2	2		27,430,267		
Total			101,250,841	204,663	69,616,088	22,843,566	4,598,751	3,987,773	Aver. 2.55 cents		3,971,423,311	947,700,000	

¹ Disposition data is estimated in some cases, and approximately allocated according to motor fuel laws.

² Actual amount available for disposal excluding refunds.

³ On State highway bonds, \$78,028; remainder on District road bonds.

⁴ New 3-cent tax effective July 29, 1927.

⁵ Paid from oil-inspection fund.

⁶ New 5-cent tax effective July 1, 1927.

⁷ New 2-cent tax effective Aug. 1, 1927.

⁸ New 3-cent tax effective July 4, 1927.

⁹ Paid from State general fund, \$4,500.

¹⁰ Collection cost of \$3,750 from Legislative appropriation.

¹¹ Includes \$125,742 for elimination of grade crossings.

¹² Assigned to Baltimore City, includes \$31,435 for elimination of grade crossings.

¹³ State highway bond retirements.

¹⁴ New 3-cent tax effective Sept. 4, 1927.

¹⁵ Collection costs, \$2,300 from inspection oil fund.

¹⁶ For sea wall to protect highway; collected by an extra tax of 2 cents in Harrison County and 1 cent in Hancock County.

¹⁷ New 2-cent tax effective July 1, 1927.

¹⁸ Includes \$4,402 for license fees for gasoline distributing stations.

¹⁹ Approximated by prorating motor vehicle fees and gasoline taxes for interest on road bonds.

²⁰ For maintenance of streets in municipalities.

²¹ Paid from general fund; approximately \$40,000.

²² New 3-cent tax effective July 1, 1927.

²³ Legislative appropriation \$5,000 for the year.

²⁴ State appropriation of \$5,000 for the year.

²⁵ New 4-cent tax effective July 1, 1927.

²⁶ For free-school fund.

²⁷ Not available for road use until 1928.

²⁸ On State road bonds.

²⁹ Costs of \$3,228 paid from appropriation from State revenues.

³⁰ Collection costs and interest of State road bonds chargeable to this fund but amounts not reported.

³¹ New 4-cent tax effective July 1, 1927.

³² Appropriation by State of \$5,000 for half year.

³³ Excludes \$600 costs not charged to tax.

³⁴ Assigned by law for use on Washington streets.

ROAD PUBLICATIONS OF BUREAU OF PUBLIC ROADS

Applicants are urgently requested to ask only for those publications in which they are particularly interested. The Department can not undertake to supply complete sets nor to send free more than one copy of any publication to any one person. The editions of some of the publications are necessarily limited, and when the Department's free supply is exhausted and no funds are available for procuring additional copies, applicants are referred to the Superintendent of Documents, Government Printing Office, this city, who has them for sale at a nominal price, under the law of January 12, 1895. Those publications in this list, the Department supply of which is exhausted, can only be secured by purchase from the Superintendent of Documents, who is not authorized to furnish publications free.

ANNUAL REPORTS

Report of the Chief of the Bureau of Public Roads, 1924.
Report of the Chief of the Bureau of Public Roads, 1925.

DEPARTMENT BULLETINS

- No. 105D. Progress Report of Experiments in Dust Prevention and Road Preservation, 1913.
- *136D. Highway Bonds. 20c.
- 220D. Road Models.
- 257D. Progress Report of Experiments in Dust Prevention and Road Preservation, 1914.
- *314D. Methods for the Examination of Bituminous Road Materials. 10c.
- *347D. Methods for the Determination of the Physical Properties of Road-Building Rock. 10c.
- *370D. The Results of Physical Tests of Road-Building Rock. 15c.
- 386D. Public Road Mileage and Revenues in the Middle Atlantic States, 1914.
- 387D. Public Road Mileage and Revenues in the Southern States, 1914.
- 388D. Public Road Mileage and Revenues in the New England States, 1914.
- 390D. Public Road Mileage and Revenues in the United States, 1914. A Summary.
- 407D. Progress Reports of Experiments in Dust Prevention and Road Preservation, 1915.
- *463D. Earth, sand-clay and gravel. 15c.
- *532D. The Expansion and Contraction of Concrete and Concrete Roads. 10c.
- *537D. The Results of Physical Tests of Road-Building Rock in 1916, Including all Compression Tests. 5c.
- *583D. Reports on Experimental Convict Road Camp, Fulton County, Ga. 25c.
- *660D. Highway Cost Keeping. 10c.
- *670D. The Results of Physical Tests of Road-Building Rock in 1916 and 1917. 5c.
- *691D. Typical Specifications for Bituminous Road Materials. 10c.
- *724D. Drainage Methods and Foundations for County Roads. 20c.
- *1077D. Portland Cement Concrete Roads. 15c.
- *1132D. The Results of Physical Tests of Road-Building Rock from 1916 to 1921, Inclusive. 10c.

DEPARTMENT BULLETINS—Continued

- No. 1259D. Standard Specifications for Steel Highway Bridges, adopted by the American Association of State Highway Officials and approved by the Secretary of Agriculture for use in connection with Federal-aid road work.
- 1279D. Rural Highway Mileage, Income, and Expenditures, 1921 and 1922.
- 1486D. Highway Bridge Location.

DEPARTMENT CIRCULARS

- No. 94C. T. N. T. as a Blasting Explosive.
- 331C. Standard Specifications for Corrugated Metal Pipe Culverts.

MISCELLANEOUS CIRCULARS

- No. 62M. Standards Governing Plans, Specifications, Contract Forms, and Estimates for Federal Aid Highway Projects.
- 93M. Direct Production Costs of Broken Stone.
- 105M. Federal Legislation Providing for Federal Aid in Highway Construction and the Construction of National Forest Roads and Trails.

FARMERS' BULLETINS

- No. *338F. Macadam Roads. 5c.
- *505F. Benefits of Improved Roads. 5c.

SEPARATE REPRINTS FROM THE YEARBOOK

- No. *739Y. Federal Aid to Highways, 1917. 5c.
- *849Y. Roads. 5c.
- 914Y. Highways and Highway Transportation.

REPRINTS FROM THE JOURNAL OF AGRICULTURAL RESEARCH

- Vol. 5, No. 17, D- 2. Effect of Controllable Variables upon the Penetration Test for Asphalts and Asphalt Cements.
- Vol. 5, No. 19, D- 3. Relation Between Properties of Hardness and Toughness of Road-Building Rock.
- Vol. 5, No. 24, D- 6. A New Penetration Needle for Use in Testing Bituminous Materials.
- Vol. 6, No. 6, D- 8. Tests of Three Large-Sized Reinforced-Concrete Slabs Under Concentrated Loading.
- Vol. 11, No. 10, D-15. Tests of a Large-Sized Reinforced-Concrete Slab Subjected to Eccentric Concentrated Loads.

* Department supply exhausted.

UNITED STATES DEPARTMENT OF AGRICULTURE
BUREAU OF PUBLIC ROADS
STATUS OF FEDERAL AID HIGHWAY CONSTRUCTION

AS OF

OCTOBER 31, 1927

FISCAL YEAR 1928

STATES	FISCAL YEARS 1917-1927				PROJECTS COMPLETED SINCE JUNE 30, 1927				* PROJECTS UNDER CONSTRUCTION				PROJECTS APPROVED FOR CONSTRUCTION				BALANCE OF FEDERAL AID FUND AVAILABLE FOR NEW PROJECTS	STATES
	TOTAL COST		FEDERAL AID		TOTAL COST		FEDERAL AID		ESTIMATED COST		FEDERAL AID ALLOTTED		ESTIMATED COST		FEDERAL AID ALLOTTED			
	\$	MILES	\$	MILES	\$	MILES	\$	MILES	\$	MILES	\$	MILES	\$	MILES	\$	MILES		
Alabama	\$ 20,061,371.68	1,400.2	\$ 9,615,099.94	1,400.2	\$ 84,878.41	0.2	\$ 40,723.98	0.2	\$ 8,345,382.61	459.4	\$ 4,016,659.69	459.4	\$ 34,528.88	17,264.44	0.3	\$ 2,224,191.39	Alabama	
Arizona	11,809,590.70	800.8	6,447,163.27	800.8	267,759.35	14.1	192,918.61	14.1	1,448,605.72	87.7	1,086,986.44	87.7	717,686.26	158,999.73	7.3	2,928,204.24	Arizona	
Arkansas	22,337,014.63	1,560.6	9,525,192.75	1,560.6	52,245.29	0.1	26,122.64	0.1	3,613,444.42	223.9	1,980,993.63	223.9	717,686.26	158,999.73	7.3	1,292,591.25	Arkansas	
California	35,128,269.04	1,306.3	16,937,086.82	1,306.3	1,653,742.65	49.3	810,515.86	49.3	7,427,892.89	148.6	3,390,057.86	148.6	29,144.22	17,486.53	0.1	3,371,184.93	California	
Colorado	15,497,121.91	829.0	7,934,299.91	829.0	91,935.74	0.9	48,486.64	0.9	6,620,930.26	284.8	3,245,327.97	284.8	80,171.65	24,498.87	3.2	2,449,719.61	Colorado	
Connecticut	5,327,322.29	137.3	2,444,000.54	137.3	1,324,934.52	14.6	355,348.84	14.6	5,143,625.87	31.9	1,450,499.75	31.9	595,818.25	150,731.24	7.8	395,795.63	Connecticut	
Delaware	6,237,026.55	159.5	2,345,572.42	159.5	263,662.17	14.6	120,820.22	14.6	998,398.73	365.3	365,371.18	365.3	11.9	150,731.24	7.8	7,919.18	Delaware	
Florida	7,476,856.31	245.1	3,627,912.60	245.1	1,517,818.43	32.7	734,206.79	32.7	8,325,791.94	176.3	3,551,765.95	176.3	60,305.70	259,730.00	18.0	790,769.66	Florida	
Georgia	31,951,435.50	2,173.6	15,101,232.40	2,173.6	3,892,173.22	163.9	1,834,341.18	163.9	6,487,697.09	223.9	3,215,959.71	223.9	60,305.70	259,730.00	18.0	229,475.86	Georgia	
Idaho	13,225,515.45	835.5	7,075,527.16	835.5	519,938.13	57.4	365,549.79	57.4	2,447,435.27	154.8	1,455,082.31	154.8	969,517.91	562,232.94	66.9	26,427.80	Idaho	
Illinois	48,538,982.16	1,530.8	22,781,516.60	1,530.8	1,014,169.01	114.3	496,539.00	114.3	14,705,056.85	507.3	7,057,757.85	507.3	3,050,176.60	1,452,638.15	102.4	1,674,714.39	Illinois	
Indiana	23,372,717.74	732.5	11,238,568.20	732.5	449,555.13	13.3	211,986.95	13.3	17,089,350.28	518.8	8,184,818.49	518.8	241,195.05	241,195.05	32.2	254,558.31	Indiana	
Iowa	34,306,139.86	2,484.4	14,395,603.75	2,484.4	1,014,169.01	114.3	496,539.00	114.3	14,947,503.76	501.4	7,057,757.85	501.4	235,050.23	91,398.66	5.6	71,264.64	Iowa	
Kansas	27,442,061.61	1,495.2	14,730,829.48	1,495.2	686,352.16	35.9	305,659.34	35.9	15,242,211.85	807.8	6,022,427.81	807.8	534,750.84	190,280.53	47.8	282,745.82	Kansas	
Kentucky	23,215,600.53	874.9	9,510,684.75	874.9	552,359.25	27.5	262,003.09	27.5	10,075,988.40	43.7	4,759,672.09	43.7	159,222.22	84,611.10	9.2	14,774.97	Kentucky	
Louisiana	15,977,552.20	1,178.7	7,083,892.21	1,178.7	831,554.53	37.9	442,316.21	37.9	3,434,131.07	95.6	1,530,386.40	95.6	1,922,511.62	951,255.78	78.4	357,955.40	Louisiana	
Maine	10,564,562.00	357.6	4,989,452.67	357.6	83,563.95	33.2	34,561.86	33.2	2,451,811.63	44.6	1,025,985.64	44.6	391,447.59	150,459.27	13.9	1,053,393.74	Maine	
Maryland	11,730,203.93	477.8	5,534,938.27	477.8	773,728.83	91.5	158,264.74	91.5	6,402,482.70	107.7	1,749,024.45	107.7	175,422.28	70,800.00	4.7	1,793,805.66	Maryland	
Massachusetts	20,570,246.02	410.4	7,425,928.15	410.4	380,950.52	31.6	181,630.53	31.6	5,102,658.03	147.4	1,754,274.75	147.4	1,754,274.75	700,394.50	33.7	552,933.92	Massachusetts	
Michigan	45,090,598.47	3,543.5	19,045,145.67	3,543.5	3,892,173.22	283.5	1,834,341.18	283.5	7,682,236.52	390.4	2,335,295.50	390.4	981,273.24	21,000.00	46.3	112,676.33	Michigan	
Minnesota	18,331,230.75	1,314.1	9,004,294.62	1,314.1	933,468.54	63.6	492,884.75	63.6	6,494,923.72	340.2	3,138,235.78	340.2	275,036.87	137,547.93	23.4	682,933.92	Minnesota	
Mississippi	42,399,290.41	1,944.8	19,591,056.48	1,944.8	2,160,732.60	76.3	999,852.79	76.3	8,995,902.95	286.4	3,912,657.34	286.4	1,821,650.41	1,047,190.66	195.4	898,054.39	Mississippi	
Missouri	12,854,995.72	1,151.5	205,458.10	1,151.5	205,458.10	25.3	152,882.58	25.3	3,433,091.18	284.6	2,425,302.78	284.6	1,821,650.41	1,047,190.66	195.4	4,053,725.28	Missouri	
Montana	16,157,040.25	2,246.6	7,739,386.39	2,246.6	2,131,360.33	203.1	1,022,276.51	203.1	13,491,924.63	134.9	6,545,818.21	134.9	793,561.66	396,795.25	73.8	416,095.64	Montana	
Nebraska	10,421,349.31	853.6	217,454.75	853.6	217,454.75	49.5	188,218.20	49.5	1,691,900.15	171.6	1,469,580.05	171.6	318,330.87	279,239.82	17.8	218,539.25	Nebraska	
Nevada	5,621,391.49	1,515.0	1,289,680.76	1,515.0	259,680.76	41.3	542,989.32	41.3	12,520,057.18	305.4	4,844,337.74	305.4	3,765,702.00	1,442,245.00	79.2	2,333,075.18	Nevada	
New Hampshire	22,228,240.08	254.8	7,778,958.05	254.8	127,942.33	4.4	62,814.27	4.4	1,515,472.17	45.0	562,133.15	45.0	70,803.54	28,110.00	1.9	3,131.53	New Hampshire	
New Jersey	13,336,250.94	1,505.2	7,495,354.48	1,505.2	1,584,371.23	24.2	362,265.00	24.2	6,812,108.77	100.8	1,544,398.56	100.8	1,544,398.56	28,110.00	1.9	2,96	New Jersey	
New Mexico	54,193,085.44	1,439.3	21,693,595.65	1,439.3	1,922,486.89	19.1	116,166.12	19.1	3,185,543.90	233.7	2,485,129.33	233.7	6,874,300.00	1,395,347.50	89.2	1,619,255.49	New Mexico	
New York	15,295,843.21	1,450.1	7,746,293.68	1,450.1	792,233.03	39.2	454,898.15	39.2	3,658,948.76	87.9	1,740,015.52	87.9	248,220.85	121,500.00	21.0	595,245.17	New York	
North Carolina	35,881,558.55	2,715.6	17,941,457.48	2,715.6	1,791,457.48	166.2	1,050,133.96	166.2	5,516,107.45	389.4	2,743,433.61	389.4	452,454.23	215,663.74	67.1	158,035.01	North Carolina	
North Dakota	5,621,391.49	1,515.0	1,289,680.76	1,515.0	259,680.76	41.3	542,989.32	41.3	12,520,057.18	305.4	4,844,337.74	305.4	3,765,702.00	1,442,245.00	79.2	2,333,075.18	North Dakota	
Ohio	30,331,957.08	1,268.1	14,117,579.21	1,268.1	1,269,742.25	9.6	379,754.99	9.6	4,339,116.17	308.4	1,853,199.24	308.4	2,600,590.55	1,308,471.72	131.1	152,672.84	Ohio	
Oklahoma	19,573,174.72	1,055.0	10,041,482.94	1,055.0	1,801,963.43	0.2	922,930.50	0.2	2,458,419.01	372.2	1,292,493.27	372.2	2,405,709.02	111,744.98	4.9	523,474.31	Oklahoma	
Oregon	77,725,194.76	1,534.3	26,317,650.32	1,534.3	1,857,836.31	45.1	577,970.03	45.1	19,420,748.76	742.1	5,927,068.82	742.1	785,457.49	328,638.97	39.8	1,086,339.34	Oregon	
Pennsylvania	19,573,174.72	1,055.0	10,041,482.94	1,055.0	1,801,963.43	0.2	922,930.50	0.2	2,458,419.01	372.2	1,292,493.27	372.2	2,405,709.02	111,744.98	4.9	523,474.31	Pennsylvania	
Rhode Island	5,233,413.39	116.0	433,553.96	116.0	433,553.96	9.9	129,827.33	9.9	1,292,493.27	372.2	1,292,493.27	372.2	2,405,709.02	111,744.98	4.9	523,474.31	Rhode Island	
South Carolina	17,002,039.93	1,568.4	7,526,999.90	1,568.4	1,599,570.97	761	734.70	39.3	8,073,707.02	261.2	2,397,460.73	261.2	735,930.03	55,938.94	5.2	114,247.80	South Carolina	
South Dakota	19,262,063.24	2,502.9	9,507,526.54	2,502.9	153,619.70	42.1	81,737.63	42.1	4,542,060.49	234.8	2,448,199.10	234.8	558,355.04	250,058.06	99.2	99,333.57	South Dakota	
Tennessee	24,233,035.03	868.7	11,551,467.55	868.7	1,449,112.56	24.6	532,743.87	24.6	9,484,114.10	443.8	3,955,685.21	443.8	554,634.68	275,334.83	35.7	479,135.74	Tennessee	
Texas	78,130,246.17	5,468.4	31,565,560.48	5,468.4	5,490,539.30	197.6	2,483,957.84	197.6	13,001,316.83	435.8	5,955,382.98	435.8	4,504,082.00	1,873,635.66	137.7	3,540,566.88	Texas	
Utah	5,037,119.23	152.7	2,348,079.95	152.7	702,679.70	11.9	268,993.49	11.9	2,656,156.63	130.5	2,003,007.34	130.5	727,135.19	197,957.00	37.9	197,957.00	Utah	
Vermont	15,184,505.97	711.1	4,573,788.01	711.1	847,151.85	5.3	415,733.32	5.3	2,735,034.21	99.7	1,003,794.57	99.7	130,902.52	79,535.23	5.0	12,487.33	Vermont	
Washington	26,944,025.24	1,199.9	12,537,143.25	1,199.9	772,796.37	8.2	117,551.39	8.2	5,406,538.94	213.1	2,193,151.16	213.1	1,370,669.79	489,000.00	58.5	795,419.73	Washington	
West Virginia	10,424,847.32	419.4	4,573,788.01	419.4	2,114,097.29	111.7	1,053,613.32	111.7	7,943,945.40	331.9	2,829,734.05	331.9	1,500,484.17	617,475.22	48.7	25,193.72	West Virginia	
Wisconsin	27,891,502.13	1,729.5	11,847,889.90	1,729.5	1,729,500.00	4.7	1,053,613.32	4.7	9,531,964.00	331.9	2,829,734.05	331.9	1,500,484.17	617,475.22	48.7	1,978,747.91	Wisconsin	
Wyoming	12,650,712.15	1,315.9	7,139,257.05	1,315.9	193,899.07	24.3	143,801.81	24.3	2,910,025.46	224.1	1,851,296.04	224.1						

* Includes projects reported completed (final vouchers not yet paid) totaling: Estimated cost \$ 117,679,214.31 Federal aid \$ 50,334,503.28 Miles 4,703.1

PUBLIC ROADS

A JOURNAL OF HIGHWAY RESEARCH



UNITED STATES DEPARTMENT OF AGRICULTURE
BUREAU OF PUBLIC ROADS



VOL. 8, NO. 10



DECEMBER, 1927



A VERMONT GRAVEL ROAD

PUBLIC ROADS

A JOURNAL OF HIGHWAY RESEARCH

U. S. DEPARTMENT OF AGRICULTURE

BUREAU OF PUBLIC ROADS

CERTIFICATE: By direction of the Secretary of Agriculture, the matter contained herein is published as administrative information and is required for the proper transaction of the public business

The reports of research published in this magazine are necessarily qualified by the conditions of the tests from which the data are obtained. Whenever it is deemed possible to do so, generalizations are drawn from the results of the tests; and, unless this is done the conclusions formulated must be considered as specifically pertinent only to the described conditions

VOL. 8, NO. 10

DECEMBER, 1927

R. E. ROYALL, Editor

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Owing to the necessarily limited edition of this publication it will be impossible to distribute it free to any persons or institutions other than State and county officials actually engaged in planning or constructing public highways, instructors in highway engineering, periodicals upon an exchange basis, and Members of both Houses of Congress. At the present time names can be added to the free list only as vacancies occur. Others desiring to obtain "Public Roads" can do so by sending 10 cents for a single number or \$1 per year to the Superintendent of Documents, U. S. Government Printing Office, Washington, D. C.

DIGEST OF VERMONT HIGHWAY TRANSPORTATION SURVEY¹

Reported by J. GORDON McKAY, Chief of the Division of Highway Economics, United States Bureau of Public Roads

THE State Highway Department of Vermont was established in 1898 to act in an advisory capacity to the towns. In 1906 the principle of State aid was adopted and the State highway department was authorized to assume the supervision of the construction and maintenance of the main thoroughfares upon which State funds were expended in conjunction with funds of the towns.²

From 1906 to 1923 the State-aid system was improved almost exclusively with gravel surfaces. Between 1923 and 1926, besides adding to the mileage of gravel roads, the surfaces of this type on the more heavily traveled routes have been surface-treated, and 36 miles of surfaces superior to gravel have been constructed.

The rapid increase in traffic on the principal highways during the past few years has materially increased the cost of maintaining gravel surfaces on these routes; and in 1925 the maintenance of roads and bridges accounted for approximately 53 per cent of the total expenditure on the State-aid system.

The principal highway problems now confronting the State are the construction of surfaces superior to gravel on the principal routes, replacing the present gravel sections upon which the maintenance has become too expensive; the construction of adequate bridges on these routes to replace the remaining old, inferior structures; the establishment of a primary highway system, including the principal routes, for which the State will accept the full responsibility of financing, construction, and maintenance; and the establishment of a secondary system of highways, including routes supplementary to the primary system to be developed under the control of the State on the State-aid principle in cooperation with the towns.

Recognizing the need for a definite program of highway improvement in accordance with the present and expected future traffic importance of the various sections of State highways, the Vermont Highway Board entered into an agreement with the United States Bureau of Public Roads to conduct a cooperative survey of transportation on the highways of the State.

DETAILED TRAFFIC DATA SECURED

The survey was begun on July 16, 1926, and continued for a period of three months.

During this period traffic data were recorded on two days each month at 149 points on the Vermont highways. Each operation consisted of a 10-hour observation period alternating between 6 a. m. to 4 p. m. and 10 a. m. to 8 p. m. Special observers tabulated traffic between 8 p. m. and 6 a. m. Complete 24-hour observations also were made to serve as the basis of a computation of hourly variation in traffic and of average daily traffic; and finally, traffic observations for one-week periods were also made at selected stations to determine variations in traffic by days of the week.

¹ This survey was conducted cooperatively by the Bureau of Public Roads of the U. S. Department of Agriculture and the Vermont State Highway Department. A copy of the full report may be obtained by addressing the Bureau of Public Roads.

² The town is the local unit of government in Vermont.

The data obtained at each of the observation points³ included counts of passenger cars, motor trucks, motor buses, horse-drawn vehicles and vehicles carrying foreign (extra-State) registration tags. Detailed motor-truck and passenger-car data were also recorded at each station, the former including the capacity of the truck, State of registration, place of ownership, origin and destination of trip, type of origin and destination, commodity carried, and tire equipment. Gross and rear-axle weights of trucks were also measured by means of portable scales. Passenger-car data included State of registration, place of ownership, purpose of trip, origin, destination, and number of passengers.

Traffic was observed on all sections of the more important traffic routes, including practically all the numbered routes and many other State-aid roads and on representative sections of the town roads. Stations



INTERVIEWING THE DRIVER OF A GROCERY TRUCK AT A WEIGHING STATION

were located so as to enable close observation of variations in traffic on the several routes and sections of routes.

DENSITY OF TRAFFIC ON ALL IMPORTANT ROUTES DETERMINED

There are 14,900 miles of highway in Vermont, of which 14,582 miles are open to public travel. Of this mileage 4,462 miles have been selected as the more important highways in each town and are known as State-aid roads, the remaining 10,120 miles being town roads.

Of the 4,462 miles of State-aid roads 1,968 miles have been numbered and marked by the State highway department. These include the more important of the State-aid roads; and the Federal-aid highway system of 1,043 miles includes the more important of the numbered routes.

This classification of Vermont highways is used throughout the report and the several classes are referred to invariably by the following names:

Federal-aid system, 1,043 miles.

Numbered State-aid routes (including all numbered State-aid routes other than the Federal-aid routes), 925 miles.

Unnumbered State-aid routes (including all State-aid roads which have not been numbered by the State highway department), 2,494 miles.

Town roads, 10,120 miles.

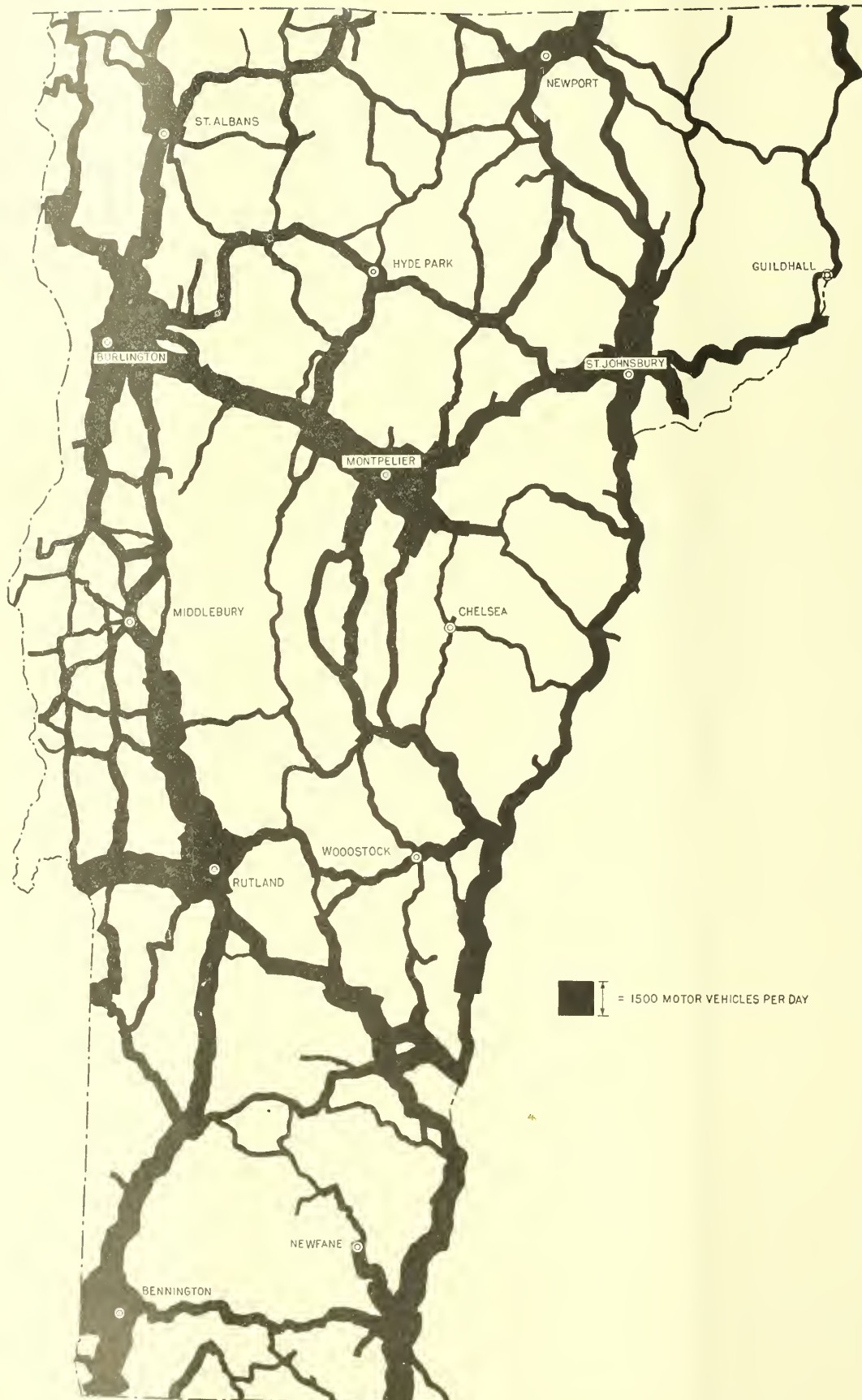


FIG. 1.—AVERAGE DAILY TRAFFIC DENSITY ON VERMONT HIGHWAYS

Upon the 14,582 miles of highway it is estimated that there was an average daily motor-vehicle movement during the period of the survey of 1,516,000 vehicle-miles.³ The average daily density of traffic is shown in Figure 1.

The State roads, comprising 30.6 per cent of the total highway mileage, carried 87.2 per cent of this total traffic; and the town roads, embracing 69.4 per cent of the total mileage, carried only 12.8 per cent of the total traffic. The traffic on the State-aid roads

Table 1 shows the mileage of Federal-aid and numbered State-aid routes by traffic classes in 1926.

The variation on individual sections of these routes is even greater. On the Federal-aid system, traffic varies from 2,673 vehicles daily on the section of heaviest traffic to approximately 100 vehicles on the least important sections. On the numbered State-aid routes, other than Federal-aid, and on the unnumbered State-aid routes, traffic of over 1,000 vehicles daily was observed only on very short sections immediately adja-



A SECTION OF CONCRETE PAVEMENT CONSTRUCTED WITH FEDERAL AID ON ROUTE U. S. 7 NEAR POWNAL

averages over 15 times that on the town roads; on the Federal-aid system the traffic is more than double that on the numbered State-aid routes, and more than five times that on the unnumbered State-aid roads; and the traffic on the numbered State-aid routes is more than double that on the unnumbered State-aid roads.

³ In this report certain terms, frequently used, have invariably the same meaning. These terms and their definitions are as follows:

Vehicles refers only to motor vehicles (passenger cars and trucks), exclusive of horse-drawn conveyances.

Traffic is defined as the movement to and fro of vehicles over a highway.

Density of traffic is defined as the number of motor vehicles passing any given point on a highway in a unit of time. For example, on route U. S. 2 between Montpelier and Barre the average daily density of traffic was 2,576 vehicles, which means that during an average 24-hour period 2,576 vehicles passed any given point on this 2 miles of highway. Unless a different unit of time is specifically stated, density of traffic refers to the number of vehicles passing any given point on the highway during a day of 24 hours. The accuracy of the determination of density of traffic is influenced by the distance between the survey stations. Exactness of method would require a density record for each point on the highway system where traffic varies. The cost involved in proportion to the relatively small gain in accuracy does not justify location of traffic observation points at close intervals. The density computed for each station on the Vermont highway system is applied to the short sections of highway reasonably adjacent to each station on which there is but little variation in traffic. In discussions of the utilization of the highway system, where it is desired to discriminate between the use of the highway by vehicles and the volume of traffic, the term *vehicle-miles per mile* is used in the former connection. Numerically, *vehicle-miles per mile* are equivalent to density of traffic.

Daily refers to a day of 24 hours.

Average daily refers to an average day during the period of the survey (July 16 to October 15, 1926).

Vehicle-mile is defined as the movement of a motor vehicle 1 mile.

Average daily vehicle-miles on the highway system are calculated by multiplying the average daily density of traffic on each section of highway by the length of the section in miles and adding the products.

Ton-mile is defined as the movement of a ton 1 mile.

Net tonnage refers to the net weight of the motor truck cargo.

Gross tonnage or *gross load* refers to the weight of the loaded motor truck, cargo and vehicle.

Foreign traffic or vehicles refers to vehicles having other than Vermont license tags. Foreign vehicle-miles are calculated by applying the percentage of foreign vehicles at each station to the total vehicle-miles on the sections of highway adjacent to each station and adding to obtain the total foreign vehicle-miles. Similar procedure is used in calculation of farm and city, business and nonbusiness, touring traffic, and trucking for hire.

cent to the larger centers of population. The minimum traffic observed on the numbered routes was 33 vehicles; on the unnumbered roads several sections were observed which carried less than 10 vehicles per day. Of the town roads a considerable number of roads observed carry less than 5 vehicles per day, and very few were observed to carry over 100 vehicles per day. The comparatively small mileage of town roads carrying over 100 vehicles per day is found in the immediate vicinity of villages. There is a considerable mileage of town roads which, although officially public roads, are unopened and carry no traffic.

In the complete report the State is divided into six traffic sections which are somewhat comparable with the distribution of population and industry, and the traffic is analyzed on the various classes of highways in each of these sections.

TABLE 1.—Mileage of Federal-aid and numbered State-aid routes, by traffic classes—1926

Traffic class (vehicles per day)	Federal-aid routes		Numbered State-aid routes	
	Miles	Per cent	Miles	Per cent
Over 1,500.....	37	3.6		
1,200 to 1,499.....	64	6.1		
800 to 1,199.....	207	19.8	10	1.1
400 to 799.....	513	49.2	190	20.5
Less than 400.....	222	21.3	725	78.4
Total.....	1,043	100.0	925	100.0

FOREIGN TRAFFIC INCREASES COST OF HIGHWAY SERVICE

Foreign traffic, i. e., traffic of vehicles registered in other States, forms an important part of the total traffic on Vermont highways. The State is traversed by the main routes of tourist traffic between southern New England and Canada and between New York and the White Mountains and Maine coast resorts, and is in itself an important recreational area.

During the period of the survey, motor vehicles of foreign registration made up 35.6 per cent of the total traffic on the Federal-aid and numbered State-aid highways. Of the total passenger-car traffic, 36.6 per cent was of foreign registration, and the corresponding percentage for truck traffic was 9.6 per cent.



ROUTE U. S. 5 NEAR HARTLAND. THE ROAD IS SURFACED WITH GRAVEL. NOTE THE STANDARD CAUTION SIGN

Foreign-truck traffic is small in volume, and largely limited to the areas adjacent to the State boundaries. In capacity and loading the foreign-truck traffic is similar to Vermont truck traffic.

Of the 1,043 miles of Federal-aid highways, 114 miles carried foreign passenger-car traffic in excess of 500 vehicles daily, 513 miles carried between 200 and 500 foreign vehicles, and 416 miles less than 200 foreign vehicles.

Route U. S. 7, from the Massachusetts line near Pownal to the Canadian border near Highgate Springs, carried the greatest volume of foreign passenger-car traffic, such traffic being 43.3 per cent of the total passenger-car traffic on the route. The large volume of foreign traffic on Vermont highways adds considerably to the cost of providing highway service on the main routes of travel. This volume of foreign traffic, in addition to local Vermont traffic, results in increased maintenance costs on present improvements, which are loaded beyond their economic capacity, and makes necessary earlier improvement or reconstruction of these routes by the construction of surfaces superior to gravel. The present contribution of foreign traffic to Vermont highway revenue is limited very largely to that derived from the taxation of gasoline sold to operators of foreign cars and it is doubtful if this revenue is at all commensurate with the increased cost of providing highway service caused by foreign traffic.

TRUCKING IMPORTANT ONLY ON SHORT SECTIONS OF THROUGH ROUTES

Except on a comparatively small part of the highway mileage, motor-truck traffic on the highways of the State is a minor part of total motor-vehicle traffic. The construction of surfaces to carry passenger cars

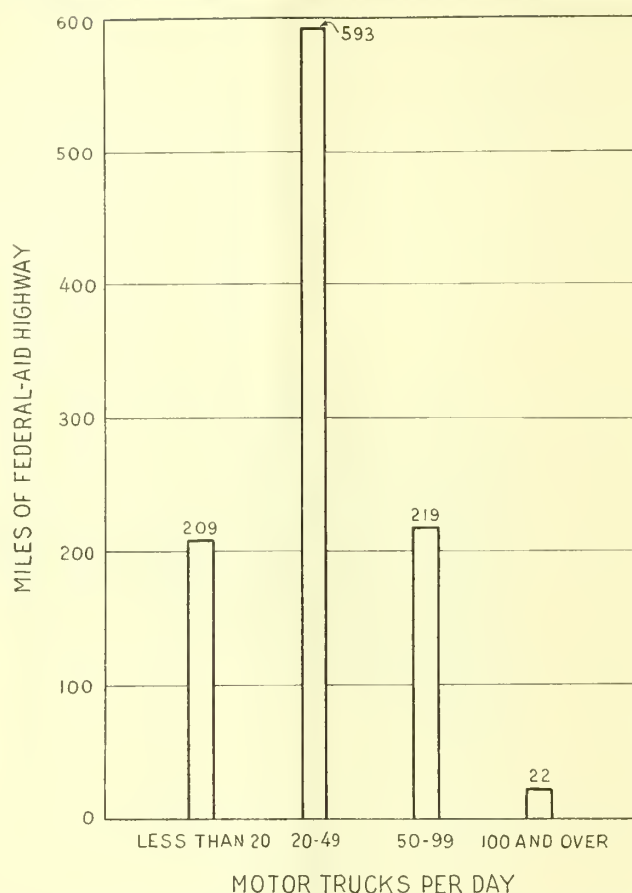


FIG. 2.—CLASSIFICATION OF FEDERAL-AID ROAD MILEAGE ACCORDING TO THE NUMBER OF MOTOR TRUCKS CARRIED

will in the majority of cases be adequate for such motor-truck traffic as there is. There are, however, major-traffic routes, particularly those near the larger cities and villages, on which the density of truck traffic is sufficiently great to require special consideration in highway planning.

There is a large mileage of road that carries less than 50 trucks a day, even on the Federal-aid system, as shown by Figure 2. Of the 1,043 miles of the latter system, only 241 carry 50 or more trucks a day; and of the 925 miles of numbered State-aid routes, only 48 miles carry truck traffic of that density.

Approximately 35 per cent of the mileage of the Federal-aid and numbered State-aid highways carried less than 20 trucks per day, and 85.3 per cent less than 50 trucks. On 13.6 per cent of the mileage there was a density of from 50 to 99 trucks, and on 1.1 per cent 100 or more trucks per day.

No one route stands out as a main trucking route. There are short sections on practically every important highway route which carry a considerable number of trucks. These sections are distributed throughout the State. The longest is on Route U. S. 2 from Burlington to East Barre, a distance of 37.9 miles, on which motor-truck traffic varied from 60 to 161 trucks at various points.

Practically all of the important trucking routes in the State are included in the Federal-aid system; and of the Federal-aid mileage carrying 50 or more trucks per day, approximately two-thirds is on the United States routes.

CAPACITY AND LOADING OF TRUCKS ANALYZED

The provision of highway facilities for motor-truck traffic in Vermont involves varied problems according to the capacity and weight of trucks as well as the density of the traffic. On roads where few or no trucks of 2 tons or larger capacity were found, highway planning need make no other provision than that required for passenger-car traffic. Trucks of less than 2 tons capacity, 97 per cent of which are equipped with pneumatic tires on their rear wheels,⁴ have much the same effect upon highway surfaces as passenger cars.

Where trucks of 2 tons or greater capacity occur in appreciable numbers they must be given consideration in highway planning. These trucks carry considerably heavier loads than passenger cars and approximately 53 per cent of them are equipped with other than pneumatic tires on the rear axle.

Figure 3 shows the distribution by capacity groups of loaded motor trucks observed on the highways of the State.

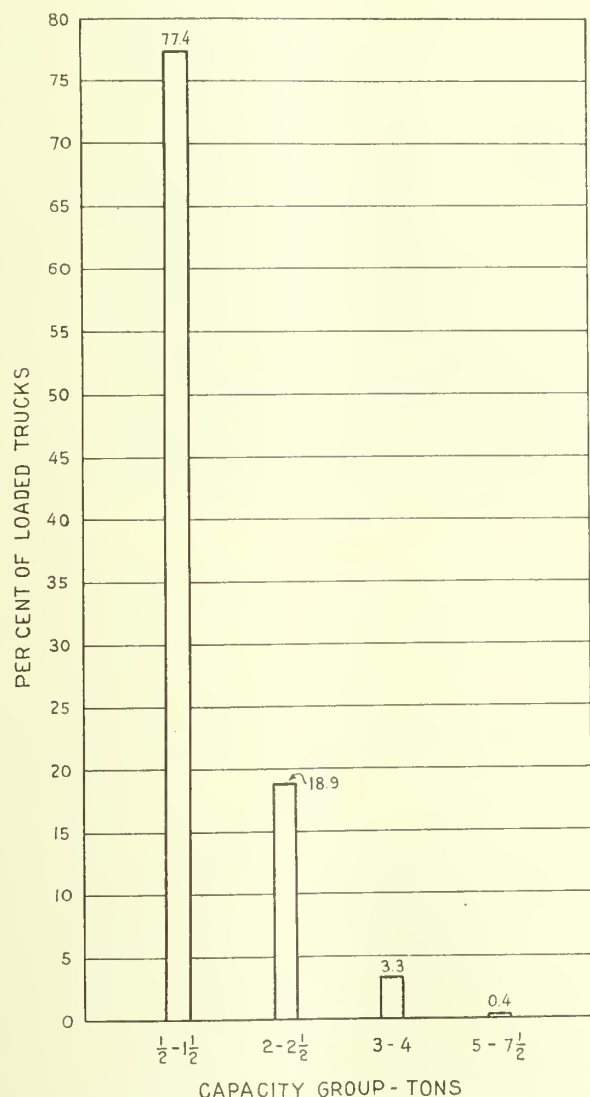


FIG. 3.—DISTRIBUTION OF LOADED MOTOR TRUCKS BY CAPACITY CLASSES

⁴ The rear axle of a truck delivers a greater impact to the road surface because it carries, on an average in Vermont, 69 per cent of the total gross load.

Motor trucks of 2 to $2\frac{1}{2}$ tons capacity were approximately one-fifth of the observed trucks, those of less than 2 tons capacity 77.4 per cent, those of 3 to 4 tons capacity comprised only 3.3 per cent, while those of 5 tons or greater capacity, being only 0.4 per cent of the total number of trucks observed, were a negligible part of the total truck traffic.



AN ATTRACTIVE VISTA ON THE CONNECTICUT RIVER ROAD. THE ROAD HAS AN OILED GRAVEL SURFACE

As shown in Table 2, loaded trucks of less than 2 tons capacity carry an average cargo of 1,800 pounds and have an average gross load of 5,140 pounds.

TABLE 2.—Average weight per loaded truck by capacity classes

Capacity class	Loaded trucks	Average net weight of cargo	Average gross weight
Tons:	Number	Pounds	Pounds
$\frac{1}{2}$ to $1\frac{1}{2}$	2,523	1,800	5,140
2 to $2\frac{1}{2}$	524	4,920	12,410
3 to 4.....	112	5,490	15,100
5 to $7\frac{1}{2}$	4	8,200	20,300

That motor-truck traffic in Vermont is predominantly a movement of small trucks carrying light loads is further indicated by the fact that 72.8 per cent of the loaded trucks weigh less than 8,000 pounds gross and 87.4 per cent less than 12,000 pounds gross. Only 3.9 per cent weigh 16,000 or more pounds, as shown in Table 3.

TABLE 3.—*Loaded trucks classified by gross weight*

Gross weight	Loaded trucks		Average net weight of cargo	Average gross weight
	Number	Per cent	Pounds	Pounds
Thousand pounds:				
Less than 4.....	869	27.5	630	3,060
4 to 8.....	1,431	45.3	2,000	5,640
8 to 12.....	463	14.6	3,600	9,580
12 to 16.....	276	8.7	5,680	13,800
16 and over.....	124	3.9	9,030	18,280
Total.....	3,163	100.0	2,450	6,710

Trucks of over 16,000 pounds gross weight carry an average cargo of 9,030 pounds and have an average gross weight of 18,280 pounds.



THE RIVER ROAD BETWEEN FARLEY AND BRADFORD—A GRAVEL ROAD CONSTRUCTED WITH FEDERAL AID

Extensive use of trucks of 2 tons capacity or larger is limited to a relatively small mileage of the highways of the State. On Federal-aid highways, the principally traveled routes of the State, less than 10 trucks of 2 tons or larger capacity are found on 68.4 per cent of the mileage as shown in Table 4.

TABLE 4.—*Mileage of Federal-aid and other numbered State-aid highways by density of traffic of 2-ton or larger trucks*

Two-ton and larger trucks per day	Federal-aid highways		Other numbered State-aid highways	
	Miles	Per cent	Miles	Per cent
0 to 9.....	713	68.4	839	90.7
10 to 19.....	284	27.2	59	6.4
20 to 29.....	36	3.4	21	2.3
30 and over.....	10	1.0	6	.6
Total.....	1,043	100.0	925	100.0

MOTOR-BUS AND MOTOR-TRUCK LINES ARE OF INCREASING IMPORTANCE

Motor-bus traffic, although small in total volume, is important on certain of the Federal-aid and numbered State-aid routes, and on a few of the unnumbered State-aid roads.

In 1926 there were 52 companies or individuals licensed by the public service commission, engaged in intrastate and interstate bus transportation, operating on regular schedules over approximately 671 miles of Federal-aid roads, 168 miles of numbered State-aid and 62 miles of unnumbered State-aid routes.

On portions of several of the main routes two or more bus lines operate, some being through lines and others serving the local communities. Burlington,

Rutland, Montpelier, and White River Junction are the principal termini of motor-bus transportation lines.

The busses observed varied in capacity from 5 to 38 passengers each; approximately one-third had a capacity of less than 10, and one-half a capacity of 20 or more passengers.

Busses of small passenger capacity are similar to the ordinary passenger automobile and require no special consideration, but the large-capacity bus traveling at high speed may, when present in large numbers, require special consideration both as to width of surface and design of pavement.

In 1926 there were 16 licensed trucking concerns operating over fixed routes and on regular schedules. These routes covered approximately 346 miles of Federal-aid routes, 74 miles of numbered State-aid routes, and 17 miles of unnumbered State-aid roads.

The capacities of trucks engaged in this form of common-carrier transportation varied from 1 to 2½ tons.

While the tonnage hauled by companies operating for hire is small compared to the tonnage of the total truck traffic, it is of growing importance in those sections which have inadequate railroad transportation.

SURVEY SHOWS EXTENT OF HIGHWAY USE

During the period of the survey, motor-vehicle traffic on the highways of the State, of which there are 14,582 miles, was approximately 139,472,000 vehicle-miles, an average of 1,516,000 vehicle-miles per day. The distribution of this traffic by classes of highway—Federal-aid, other numbered State-aid routes, unnumbered State-aid routes, and town roads—is shown in Table 5.

TABLE 5.—*Motor vehicle utilization and mileage of Vermont highways by systems*

Highway system	Highway mileage		Average daily vehicle-mileage		Average daily density of traffic
	Miles	Per cent	Vehicle-miles	Per cent	
Federal-aid system.....	1,043	7.2	732,000	48.3	702
Numbered State-aid routes.....	925	6.3	271,000	17.9	293
Unnumbered State-aid routes.....	2,494	17.1	318,000	21.0	128
Town roads.....	10,120	69.4	195,000	12.8	19
Total.....	14,582	100.0	1,516,000	100.0	

¹ Numbered State-aid routes other than Federal-aid routes. The numbered State-aid routes in 1926 included the Federal-aid system of 1,043 miles, and other numbered routes, 925 miles, a total of 1,968 miles.

The Federal-aid highway system, 7 per cent of the certified highway mileage and 7.2 per cent of the mileage of traveled public roads, carried approximately one-half of the total traffic on all roads of the State. The present numbered routes, 13.5 per cent of the highway mileage, which includes the Federal-aid system, carried two-thirds of the total traffic. Town roads, 69.4 per cent of the total highway mileage, carried only 12.8 per cent of the traffic.

The routes selected for uniform numbering by the American Association of State Highway Officials; including U. S. 2, U. S. 4, U. S. 5, and U. S. 7 are the most important through traffic routes in the State. These routes, aggregating 525 miles in length, approximately 50 per cent of the Federal-aid system, carry approximately 60 per cent of the traffic on the system.

The primary Federal-aid routes, 446 miles, three-sevenths of the Federal-aid system, carry 53 per cent of the traffic on the system.

COMPOSITION OF TRAFFIC DISCUSSED

The comparative use of the highways of the State by vehicles of Vermont and foreign registration, by city and farm-owned vehicles, and by various other types of vehicles can be expressed accurately in vehicle-miles.⁵

The total passenger-car use of the Federal-aid and numbered State-aid roads on an average day, during the transportation survey, was 940,600 passenger-car-miles. The distribution of this traffic according to registration, ownership, type of trip, and type of usage of cars is shown in Table 6.

TABLE 6.—Composition of passenger car traffic on the Federal-aid and numbered State-aid roads

Type of passenger car traffic	Daily passenger-car-miles	Percentage of daily passenger-car-miles
State of registration:		
Vermont.....	596,300	63.4
Foreign.....	344,300	36.6
Place of ownership:		
City.....	845,600	89.9
Farm.....	95,000	10.1
Type of trip:		
Touring.....	134,500	14.3
Nontouring.....	806,100	85.7
Type of usage:		
Business.....	317,000	33.7
Pleasure.....	623,600	66.3
All types.....	940,600	100.0



STANDARD U. S. ROUTE MARKER ON U. S. ROUTE NO. 5, NEAR HARTLAND

Foreign traffic amounts to 344,300 passenger-car-miles per day, or 36.6 per cent of the total passenger-car mileage.

The traffic of farm-owned passenger cars comprises 10.1 per cent and of city-owned passenger cars, 89.9 per cent of the total passenger-car traffic on the Federal-aid and numbered State-aid routes.

The volume of farm-owned passenger-car traffic varies with the agricultural development of the area served by the routes but is more nearly uniform on all routes than is the case with city-owned traffic. On heavy-traffic routes farm-owned traffic forms a very small part of the total. On light-traffic routes off the main routes of travel and not adjacent to centers of

population, farm-owned traffic forms a much larger part of total traffic.

The importance of long-distance touring traffic on Vermont highways is indicated by the fact that 14.3 per cent of the total passenger-car traffic, measured in vehicle-miles, is made up of touring trips. This traffic is largely of foreign registration and limited to the main through routes and routes leading to points of historic or scenic interest.

Approximately two-thirds of the passenger-car traffic, measured in vehicle-miles, on the Federal-aid and other numbered State-aid routes is made up of cars used for pleasure or recreational purposes. This traffic is limited largely to the main routes of travel, the scenic routes, and routes in the summer-resort areas.

The distribution by length of trip, as shown in Table 7, is based on the entire trip from point of origin to point of destination, which—particularly for long-distance traffic—includes a considerable mileage on highways of adjacent States. The average total trip mileage and trip mileage on highways of Vermont for each type of passenger car traffic are shown in Table 8.

⁵ The various types of traffic are defined as follows:

State of registration: *Vermont* includes all motor vehicles registered in Vermont; *foreign* includes all motor vehicles not registered in Vermont.

Place of ownership: *Farm* includes all motor vehicles owned by persons residing on farms; *city* includes all motor vehicles owned by persons residing in cities, villages, or urban areas.

Type of usage: *Business* indicates that the car on the trip recorded was being used for business purposes; *pleasure* indicates that the car on the trip recorded was being used for pleasure or recreational purposes.

Type of trip: *Touring* includes all trips of more than one day's duration taken primarily for recreation; *nontouring* includes all other trips.

Type of trucking: *For hire* includes all trucks engaged in hauling commodities either on a contract or tariff basis.

TABLE 7.—*Distribution of passenger-car traffic by length of trip*¹

Length of trip Miles	Type of passenger-car traffic								
	Total	Vermont	Foreign	City	Farm	Touring	Non-touring	Business	Pleasure
	Per cent	Per cent	Per cent	Per cent	Per cent	Per cent	Per cent	Per cent	Per cent
Less than 10.....	29.8	42.0	10.8	28.1	75.6	0.0	36.3	43.0	23.1
10 to 19.....	16.1	21.9	6.9	16.1	15.0	0.0	19.6	21.6	13.3
20 to 29.....	6.9	8.7	4.1	7.0	3.7	0.0	8.4	7.9	6.4
30 to 39.....	5.0	6.1	3.1	5.1	2.7	0.0	6.1	5.0	5.0
40 to 49.....	3.8	4.5	2.7	3.9	1.0	0.0	4.6	3.8	3.8
50 to 59.....	2.4	2.6	2.0	2.5	0.0	0.4	2.8	1.7	2.7
60 to 69.....	1.9	2.1	1.6	1.9	0.3	0.5	2.2	2.1	1.8
70 to 79.....	1.8	1.8	1.8	1.8	0.7	1.1	1.9	1.6	1.8
80 to 89.....	1.5	1.3	1.8	1.5	0.0	1.0	1.6	1.3	1.6
90 to 99.....	1.4	1.2	1.7	1.5	0.0	0.8	1.5	0.9	1.7
100 to 149.....	5.3	3.6	8.1	5.5	1.0	4.2	5.6	4.6	5.7
150 to 199.....	4.0	1.4	8.1	4.1	0.0	6.4	3.5	2.4	4.8
200 to 299.....	8.4	1.8	18.8	8.8	0.0	27.6	4.3	2.8	11.3
300 and over.....	11.7	1.0	28.5	12.2	0.0	58.0	1.6	1.3	17.0
Total.....	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0

¹ Based upon a total of 8,262 cars.TABLE 8.—*Average mileage per trip and average passengers per car for various types of passenger-car traffic*

Type of traffic	Average mileage per trip ¹		Average number of passengers per car
	Total	On Vermont highways	
State of registration:			
Vermont.....	31	24	2.7
Foreign.....	199	75	3.0
Place of ownership:			
City.....	102	46	2.8
Farm.....	12	11	2.4
Type of trip:			
Touring.....	348	118	3.3
Nontouring.....	47	30	2.7
Type of usage:			
Business.....	38	25	1.9
Pleasure.....	123	52	3.2

¹ Averages shown are the arithmetic mean of the trip-mileage of cars observed. This average is influenced greatly by exceptionally long trips, but provides a reliable basis of comparing the various types of traffic.

Average daily motor-truck use of the Federal-aid and other State-aid highways during the period of the survey was 62,400 vehicle-miles. The distribution of this traffic by place of registration, ownership, and type of trucking is shown in Table 9.

TABLE 9.—*Composition of motor-truck traffic on the Federal-aid and numbered State-aid roads*

Type of truck traffic	Average daily truck-miles	Per cent of total daily truck-miles
State of registration:		
Vermont.....	56,400	90.4
Foreign.....	6,000	9.6
Type of trucking:		
For hire.....	13,900	22.2
Other than for hire.....	48,500	77.8
Place of ownership:		
City.....	50,600	81.1
Farm.....	11,800	18.9
Total.....	62,400	100.0

The proportionate use of highways in the State by trucks of foreign registration is considerably less than the use by foreign passenger cars. As shown in Table 9 only 9.6 per cent of the total motor-truck miles is produced by foreign trucks.

Foreign trucks operate primarily upon the principally traveled routes near the State boundaries. Used household furniture, fresh fruits, groceries, gasoline, and bakery goods are the principal commodities hauled by them.

Little difference was found in the average net cargo hauled by Vermont and foreign trucks, that of the former being 2,460 pounds and of the latter 2,360 pounds.

The use of the Federal-aid and numbered State-aid roads by trucks for hire totals 13,900 truck-miles per day, or 22.2 per cent of the total truck use of these highways. These trucks are engaged principally in the hauling of clay, gravel, sand, stone, milk, lumber, used household furniture, and general freight. Approximately 70 per cent of the trucks operating for hire are engaged in the transportation of these commodities. The average net cargo of trucks operated for hire is 3,300 pounds as compared with 2,240 pounds for other trucks.

City-owned trucks comprise 81.1 per cent of the motor-truck use of the same systems as shown in Table 9. The loads carried by city-owned trucks are considerably greater than those hauled by farm-owned trucks. The average net cargo hauled by city trucks is 2,710 pounds as compared with 1,270 pounds for farm-owned trucks. This small cargo hauled by farm-owned trucks and a correspondingly low gross weight indicate that the use of the highways of the State by farm-owned trucks is mainly by small-capacity trucks, hauling light loads.

That motor-truck traffic is primarily a local and short-haul movement is shown by Table 10. Of the loaded trucks observed on the principal highways of the State, 45.2 per cent were traveling less than 10 miles per trip and 78.4 per cent less than 30 miles. Of the net tonnage of commodities transported 42.7 per cent was being hauled less than 10 miles and 78 per cent less than 30 miles. Only 7.8 per cent of the net tonnage hauled by loaded trucks was transported 60 miles or more.

TABLE 10.—*Distribution of motor-truck traffic by length of trip*¹

Length of trip	Motor trucks	Net cargo tonnage
Miles:	Per cent	Per cent
Less than 10.....	45.2	42.7
10 to 19.....	22.9	25.8
20 to 29.....	10.3	9.5
30 to 39.....	6.3	6.4
40 to 49.....	5.0	5.0
50 to 59.....	2.8	2.8
60 to 69.....	1.3	1.3
70 to 79.....	1.0	1.1
80 to 89.....	.8	.7
90 to 99.....	.6	.6
100 and over.....	3.8	4.1
Total.....	100.0	100.0

¹ Based upon 3,163 loaded trucks.

The distribution by lengths of trip, as shown in Table 10, is based on the total trip from point of origin to point of destination, which—particularly for the longer trips—includes the mileage traveled on highways of adjacent States. The average total trip mileage, the trip mileage on the highways of Vermont, the average net cargo weight and the average gross weight per truck for each type of truck traffic are shown in Table 11.

TABLE 11.—Average mileage per trip and average weight per truck for various types of motor-truck traffic

Type of truck traffic	Average mileage per trip ¹		Average weight	
	Total	On Vermont highways	Net cargo	Gross
State of registration:			<i>Pounds</i>	<i>Pounds</i>
Vermont.....	19	16	2,460	6,660
Foreign.....	59	30	2,360	7,260
Type of trucking:				
For hire.....	24	13	3,300	8,040
Other than for hire.....	23	18	2,240	6,380
Place of ownership:				
City.....	26	20	2,710	7,200
Farm.....	12	11	1,270	4,440

¹ Averages shown are the arithmetic mean of trip mileage of trucks observed. This average is influenced by the relatively small number of long trips but provides a reliable basis of comparing the various types of traffic.

FUTURE TRAFFIC FORECAST

Since no adequate historical series of traffic records are available in Vermont it is impossible to make a forecast based directly upon past trends. In States where historical series of traffic records are available highway traffic and motor-vehicle registration have been found to increase at equal rates. A comparison of highway traffic and motor-vehicle registration in Maine, Maryland, Massachusetts, Michigan, and Wisconsin is shown in Figure 4.

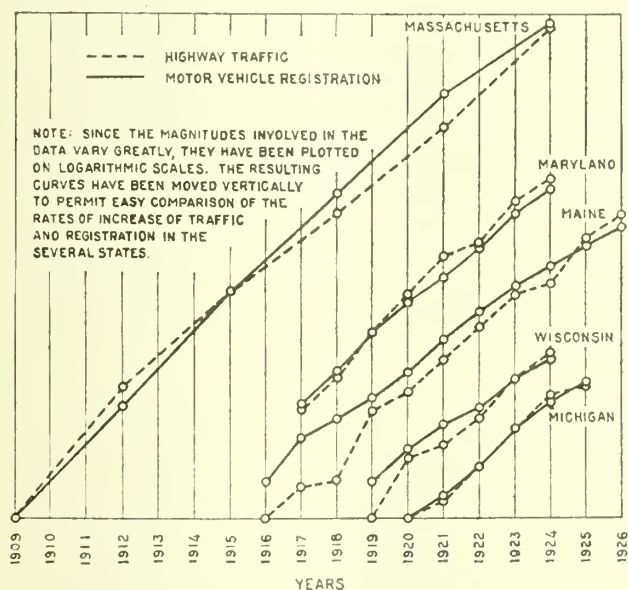


FIG. 4.—TRENDS OF HIGHWAY TRAFFIC AND MOTOR VEHICLE REGISTRATION IN MASSACHUSETTS, MARYLAND, WISCONSIN, AND MICHIGAN

Vermont varies from these States with respect to traffic growth principally in the volume of foreign traffic on the more important highways and in the rate of population increase. The proportion of foreign traffic on Vermont highways was recorded at 33 points during a traffic count conducted by the State highway department in August, 1924. At 32 of these points, which were similar in location to stations of the 1926 survey, it was found that 38.1 per cent of the cars were of foreign registration. At the 32 similarly located

stations during the 1926 survey it was found that 38.6 per cent of the traffic was foreign. In New Hampshire, which also has a large volume of foreign traffic, traffic counts obtained in 1918, 1922, and 1926 indicate that foreign traffic at corresponding points was 41 per cent of the total in 1918; 40 per cent in 1922; and 48 per cent in 1926.

These data indicate that foreign traffic in these States is increasing slightly more rapidly than local traffic and that a forecast of total traffic on Vermont highways based on motor-vehicle registration in the State would be conservative, but for a short period of years would represent total traffic with reasonable accuracy.



A BITUMINOUS MACADAM ROAD NEAR ADDISON

Motor-vehicle registration can be predicted on the basis of exact records available since 1913. The increase in motor-vehicle registration is a function of two variables (1) increase in population and (2) the increase in ownership and use of motor vehicles in proportion to population, measured by the number of persons per motor vehicle.

Population, motor-vehicle registration, and persons per car from 1913 to 1926 and extended to 1936 are shown in Table 12.

The persons per car for each year from 1913 to 1925 and the extension of the trend to 1936 are shown in Figure 5.

The trend of motor-vehicle registration in Vermont from 1913 to 1926, inclusive, indicates an increase in registration of 39.8 per cent between 1926 and 1931, and of 24.5 per cent between 1931 and 1936, or an increase of 74 per cent for the 10-year period from 1926 to 1936.⁶ Assuming a uniform rate of increase in traffic and motor-vehicle registration, traffic may be expected to increase in the same ratio as the registration.

This rate of increase in highway traffic will apply for the State as a whole. Traffic originating in the areas of the State which are rapidly increasing or de-

⁶ Based on actual registration in 1926, which is the measure of traffic for this year and estimated registration in 1931 and 1936.

TABLE 12.—Comparison of population and the number of motor vehicles in the State of Vermont

Year	Population ¹ (hundreds)	Registration of motor vehicles (hundreds)		Persons per car	
		Actual	Estimated	Actual	Estimated
1913.....	3,548	59	62	60.0	57.6
1914.....	3,544	85	85	41.8	41.6
1915.....	3,541	115	114	30.8	31.1
1916.....	3,537	157	149	22.6	23.8
1917.....	3,533	216	188	16.3	18.8
1918.....	3,530	226	233	15.6	15.1
1919.....	3,526	268	284	13.2	12.4
1920.....	3,524	316	339	11.1	10.4
1921.....	3,524	373	397	9.46	8.87
1922.....	3,524	439	459	8.03	7.68
1923.....	3,524	528	523	6.68	6.74
1924.....	3,524	612	589	5.76	5.99
1925.....	3,524	696	655	5.07	5.38
1926.....	3,524	741	722	4.76	4.88
1927.....	3,524	-----	788	-----	4.47
1928.....	3,524	-----	853	-----	4.13
1929.....	3,524	-----	916	-----	3.85
1930.....	3,524	-----	977	-----	3.61
1931.....	3,524	-----	1,036	-----	3.40
1936.....	3,524	-----	1,290	-----	2.73

¹ Population as of July 1, each year. For the years 1913 to 1923, inclusive, the populations given are Bureau of Census estimates. Those for the years 1924 to 1936, inclusive, are extensions by the method used by the Bureau of the Census.

creasing in population will increase more or less rapidly than the average. Such areas, however, are small and, considering the volume of traffic on the principal highways which originates outside of the immediate local areas, the application of a uniform rate of traffic increase to the entire State is justified.⁷

The expected traffic in 1931 was obtained by applying the rates of motor-vehicle registration and traffic increase to the 1926 traffic at each survey station. The resulting forecast of traffic at each station is shown in the full report.

ROUTES CLASSIFIED ACCORDING TO PRESENT AND FUTURE TRAFFIC

To provide a basis for the establishment of a balanced program of highway improvement to meet traffic needs in Vermont a traffic classification of Vermont high-

⁷ The validity of this assumption is substantiated by an analysis of registration increase by towns in New Hampshire for the years 1922 to 1925, inclusive. Notwithstanding variations in population density, population trends and present persons per car, the rate of decrease in persons per car in the different areas was very uniform.

ways has been established on the basis of the principal traffic data, which are (1) total present motor-vehicle traffic and estimated traffic in 1931 and 1936, (2) total truck traffic, and (3) traffic of large-capacity trucks. The highways are classified in three groups, designated as major, medium, and minor traffic routes or sections of routes, according to their average daily present and estimated future traffic.

These classes and the traffic limits of each class are summarized in Table 13:

TABLE 13.—Traffic classification of Vermont highways and traffic limits for each class for 1926 and estimated for 1931 and 1936

Traffic classification ¹	Average daily motor vehicles		
	1926	1931	1936
Major 1.....	1,500 or over.....	1,500 or over.....	1,500 or over.....
Major 2.....	800 to 1,500.....	1,500 or over.....	1,500 or over.....
Medium 1.....	800 to 1,500.....	800 to 1,500.....	1,500 or over.....
Medium 2.....	800 to 1,500.....	800 to 1,500.....	800 to 1,500.....
Medium 3.....	Less than 800.....	800 to 1,500.....	800 to 1,500.....
Minor 1.....	Less than 800.....	Less than 800.....	800 to 1,500.....
Minor 2.....	Less than 800.....	Less than 800.....	Less than 800.....

¹ The traffic classifications for 1931 and 1936 are based on average traffic in 1926.

The above traffic limits are based primarily upon experience and present practices in Vermont. The upper limit of the minor group is higher than that commonly accepted in many States, but is in accordance with Vermont traffic conditions, particularly the very limited use of trucks of over 2½ tons capacity, and the resulting absence of heavy wheel loads, the fact that observed traffic represents traffic during the period of greatest traffic density, and the serviceable type of gravel available for construction of gravel roads in Vermont. The more important routes included in the minor classification, and classified as requiring gravel surfaces, should be surface treated.

All highways upon which the traffic was observed are classified, including the entire Federal-aid system, practically all of the other numbered State-aid routes and approximately one-tenth of the unnumbered

(Continued on p. 229)

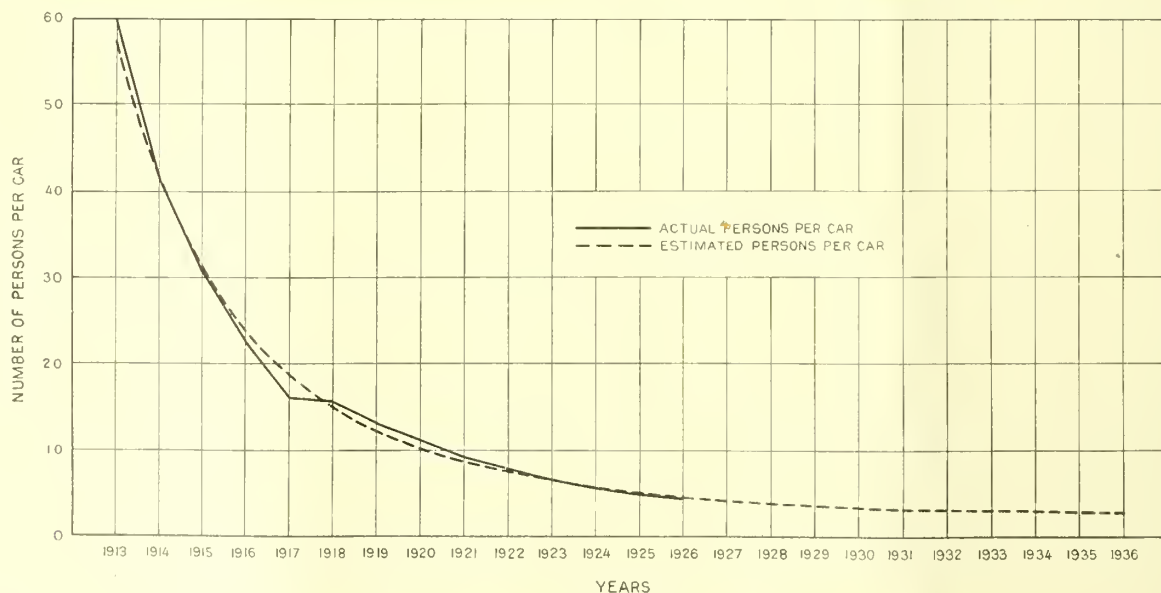


FIG. 5.—NUMBER OF PERSONS PER MOTOR VEHICLE IN VERMONT BY YEARS

RELATION BETWEEN SODIUM SULPHATE SOUNDNESS TEST AND ABSORPTION OF SEDIMENTARY ROCK

By the Division of Tests, United States Bureau of Public Roads, Reported by D. O. WOOLF, Jr., Materials Engineer

FOR a number of years past an accelerated sodium sulphate soundness test has been used to indicate the resistance of sedimentary rock to repeated frost action. This test was apparently first proposed by M. Brard, and is described in *Annales de Chimie et de Physique* (1828). It has recently been tentatively accepted by the American Association of State Highway Officials as a standard test.

The test consists of alternately immersing the test sample of rock in a saturated solution of sodium sulphate for 20 hours and drying it at 100°C. for four hours. This operation is repeated five times and then the sample is examined as to appearance and the rock classified as sound or unsound. Samples which crack, check, or disintegrate are considered as unsound and of questionable suitability for use in concrete which is exposed to frost action.

Several attempts have been made to correlate the accelerated sodium sulphate soundness test of rock with the results of other tests which are usually made



FIG. 1.—A LIMESTONE SAMPLE BEFORE AND AFTER THE SOUNDNESS TEST. THE RECTANGULAR APPEARANCE IS BECAUSE SAWED MATERIAL WAS SUBMITTED FOR TESTING AND IS NOT CHARACTERISTIC OF SAMPLES PREPARED FOR TESTING



FIG. 2.—A LIMESTONE SAMPLE BEFORE AND AFTER TESTING

to determine the quality of the material. These comparisons have usually shown negative results, i. e., there is no apparent relationship between the results of the soundness test and any of the standard tests. Considering, however, that failure in the accelerated soundness test may be caused through crystal growth in the interior of a rock specimen, it is believed that there must be some relation between the soundness test and a test of the porosity of the material. This latter property is usually determined by an absorption test, the test result expressing the amount of absorbed water in terms of percentage of the dry weight of the solid rock.

For the last three years the Bureau of Public Roads has been making the accelerated soundness test on all rock samples of sedimentary origin as well as marble which have been submitted for test to determine their suitability for use in concrete, and the tests reported in this paper include the great majority made on such rock. One hundred and fifty of these rocks have also been tested for absorption. The results of these tests are shown in Table 1, and, although they do not establish a definite relation between percentage of absorption and unsoundness as determined by the sodium sulphate test, they do indicate that rocks having a certain percentage of absorption will in the greater number of cases prove to be unsound. It is believed

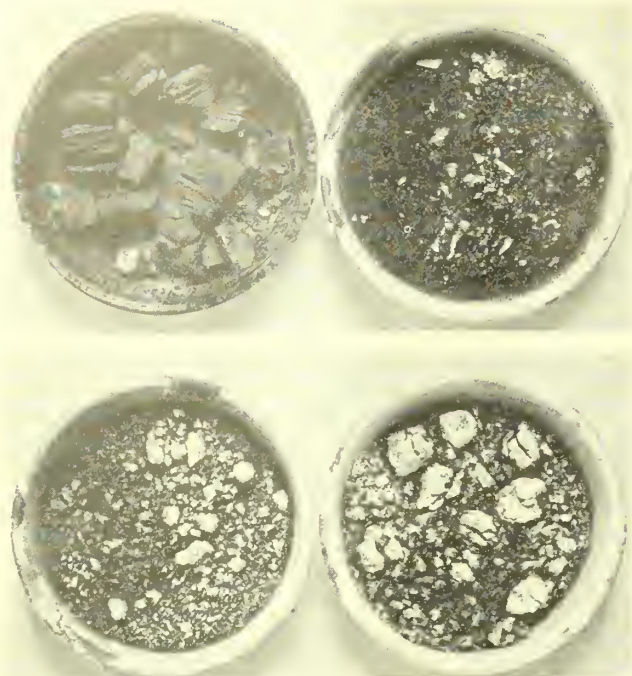


FIG. 3.—THREE SAMPLES WHICH HAVE COMPLETELY DIS-INTEGRATED IN THE SOUNDNESS TEST AND ONE SHOWING PARALLEL PARTING. ALL OF THESE SAMPLES HAD MORE THAN 4 PER CENT ABSORPTION

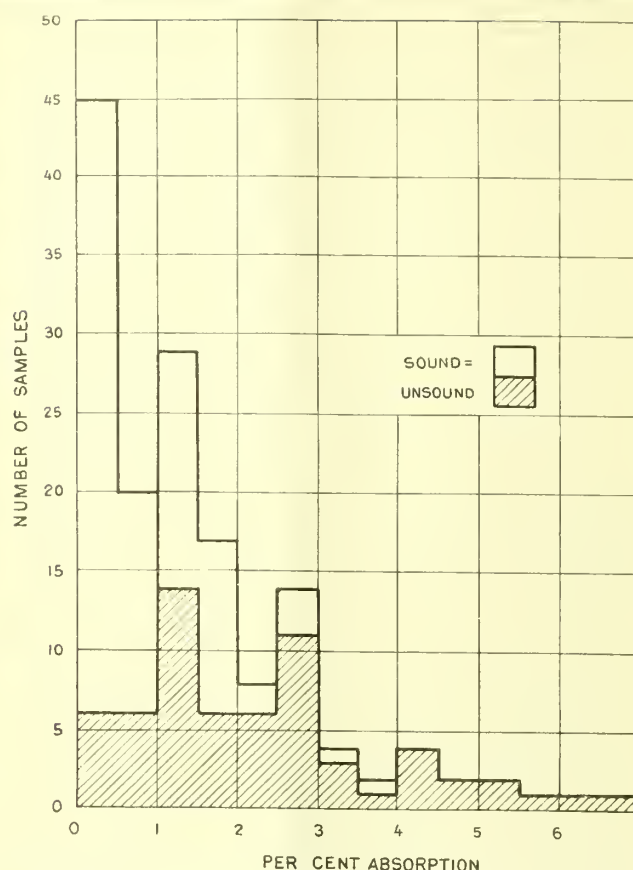


FIG. 4.—PROPORTION OF SOUND AND UNSOUND SAMPLES GROUPED ACCORDING TO PERCENTAGE OF ABSORPTION

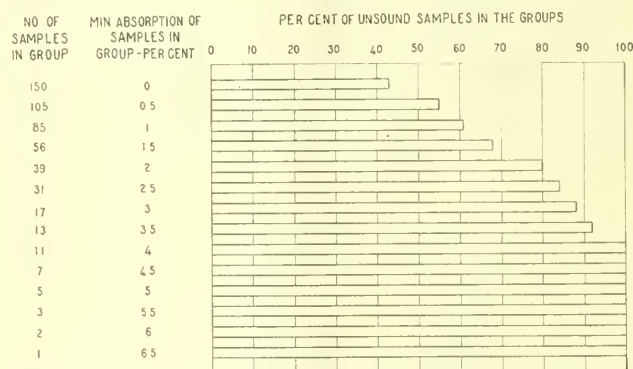


FIG. 5.—PROPORTION OF UNSOUNDNESS IN GROUPS OF SAMPLES HAVING VARIOUS MINIMUM PERCENTAGES OF ABSORPTION

that the samples shown in Table 1 include all types and qualities of sedimentary rock which are used as coarse aggregate in concrete, and the range in test values, as will be noted, is quite extensive, the absorption ranging to above 6.5 per cent. Results of the standard Deval abrasion test are not shown but they varied from a minimum of 2.3 per cent to a maximum of 26.2 per cent. The geographical distribution of the samples is shown in Table 2.

Figures 1 to 3 show typical examples of failure in the accelerated soundness test. Figures 1 and 2 show samples of unsound rock before and after the test. Figure 3 shows three completely disintegrated samples and one in which parallel parting is excellently marked. These samples (fig. 3) had very high absorptions, all of them being greater than 4 per cent.

Figure 4 shows the proportion of sound and unsound samples grouped according to percentages of absorption. Each group covers a range of 0.5 per cent, the first group including all values up to 0.5 per cent, the second from 0.51 to 1, etc. Figure 5 shows the number of samples having percentages of absorption equal to or greater than 0, 1, 2, 3, etc., and the percentage of all such groups shown to be unsound by the soundness test.

It will be noticed that of all the samples tested, 43 per cent were unsound. Of all samples tested which had an absorption of 2 per cent or more, 82 per cent were unsound. All samples with an absorption of 4 per cent or more were unsound. On the basis of the group of samples reported in this paper, the probabilities, therefore, are that two out of every five samples of sedimentary rock are unsound as determined by the sodium sulphate test.

Although the accelerated sodium sulphate soundness test is used to indicate the possibility of failure through frost action, it does not follow that failure

from freezing and thawing is always associated with unsoundness as indicated by the test. Many rocks which are seriously affected by the accelerated soundness test have been successfully used in concrete pavements and structures exposed to frost action for years.

The durability of concrete in which unsound rock was used may result from the quality of the concrete and in spite of the quality of the aggregates. A thoroughly water-tight concrete would certainly delay and possibly entirely prevent disintegration by frost action of an unsound rock used as the coarse aggregate. It is apparent that for durable concrete not only the soundness of the ingredient materials should be considered, but also the character of the concrete itself. A discussion of the soundness of concrete is not within the scope of this paper; but as the character of concrete

is variable and subject to many conditions which may affect the quality, every effort should be made to control these factors. For this reason, the use of aggregates of unquestioned durability or soundness is important. In this connection, the accelerated sodium sulphate soundness test is of value in indicating questionable aggregates. Samples which show unfavorable results with this test should be further investigated by inspection of exposed ledges or faces of the stone at the quarry, or of concrete made of the rock, if it has been previously used, before a final decision is made.

The high percentage of possible failure indicated above shows that great care should be exercised in the selection of sedimentary rock for use in concrete.

These results seem to indicate that, when time is limited, some idea of the relative behavior in the sodium sulphate soundness test of several otherwise comparable materials may be obtained from the absorption test, the highest values indicating the greatest probabilities of failure in the other test.

TABLE 1.—Results of soundness test and absorption test grouped by kinds of rock and arranged in order of the percentage of absorption

Sample No.	Location	Name	Soundness test	Absorption per cent
1	New York	Limestone	Sound	0.02
2	Maryland	do	do	.03
3	New York	do	do	.04
4	South Dakota	do	do	.05
5	Tennessee	do	Unsound	.07
6	Maryland	do	Sound	.09
7	Texas	do	do	.11
8	do	do	do	.14
9	West Virginia	do	do	.14
10	Pennsylvania	do	do	.15
11	New York	do	do	.16
12	do	do	do	.17
13	Oklahoma	do	Unsound	.17
14	New York	do	Sound	.18
15	do	do	do	.30
16	Tennessee	do	Unsound	.33
17	Ohio	do	Sound	.34
18	Florida	do	do	.52
19	Texas	do	do	.76
20	do	do	do	.77
21	Ohio	do	Unsound	.87
22	Florida	do	do	.94
23	Texas	do	do	.98
24	Indiana	do	do	1.05
25	Texas	do	do	1.10
26	Florida	do	do	1.11
27	Texas	do	Sound	1.44
28	do	do	do	1.45
29	Ohio	do	Unsound	1.68
30	Alabama	do	Sound	1.70
31	Ohio	do	Unsound	2.01
32	do	do	Sound	2.54
33	do	do	Unsound	2.65
34	Florida	do	do	3.03
35	Texas	do	do	4.12
36	West Virginia	Argillaceous limestone	Sound	.11
37	Texas	do	Unsound	.40
38	do	do	do	.52
39	Ohio	do	Sound	.59
40	Minnesota	do	Unsound	1.35
41	Texas	do	do	1.47
42	do	do	do	2.64
43	Oklahoma	do	do	2.98
44	Texas	do	do	3.36
45	Oklahoma	do	do	3.64
46	Kansas	do	do	4.06
47	do	do	do	4.21
48	do	do	do	4.85
49	do	do	do	5.20
50	Texas	do	do	5.32
51	Kansas	do	do	5.61
52	do	do	do	6.45
53	Maryland	Siliceous limestone	Sound	.02
54	do	do	do	.09
55	do	do	do	.12
56	Kansas	do	do	.15
57	Pennsylvania	do	do	.24
58	Texas	do	do	.37

TABLE 1.—Results of soundness test and absorption test grouped by kinds of rock and arranged in order of the percentage of absorption—Continued

Sample No.	Location	Name	Soundness test	Absorption per cent
59	Louisiana	Siliceous limestone	Unsound	1.05
60	Illinois	do	do	1.71
61	Kansas	do	do	2.71
62	Illinois	do	do	2.72
63	do	do	do	2.91
64	Ohio	do	do	1.40
65	do	Dolomitic limestone	Sound	2.86
66	Rhode Island	do	do	4.58
67	Missouri	Crystalline limestone	Sound	.42
68	Ohio	do	Unsound	.46
69	Texas	do	Sound	.99
70	New York	Cherty limestone	do	.08
71	do	do	do	.08
72	Illinois	do	Unsound	1.90
73	Maryland	Marble	Sound	.11
74	Louisiana	do	Unsound	.29
75	New York	Dolomitic marble	Sound	.20
76	Texas	do	do	.23
77	West Virginia	Dolomite	do	.02
78	New York	do	do	.04
79	do	do	do	.05
80	Virginia	do	do	.06
81	Pennsylvania	do	do	.14
82	New York	do	do	.21
83	Michigan	do	do	.39
84	Ohio	do	do	.39
85	do	do	do	.41
86	do	do	do	.49
87	do	do	do	.58
88	do	do	do	.64
89	do	do	do	.69
90	do	do	Unsound	.72
91	do	do	Sound	.77
92	South Dakota	do	do	.83
93	Ohio	do	do	.86
94	do	do	do	1.03
95	do	do	do	1.08
96	do	do	do	1.08
97	do	do	Unsound	1.14
98	do	do	Sound	1.19
99	do	do	do	1.27
100	do	do	do	1.27
101	do	do	do	1.28
102	Wyoming	do	do	1.32
103	Illinois	do	Unsound	1.35
104	Ohio	do	Sound	1.36
105	do	do	do	1.36
106	do	do	Unsound	1.40
107	do	do	do	1.47
108	do	do	Sound	1.52
109	do	do	do	1.59
110	Indiana	do	Unsound	1.66
111	Ohio	do	Sound	1.76
112	do	do	do	1.77
113	do	do	do	1.87
114	Indiana	do	Unsound	2.13
115	Ohio	do	Sound	2.41
116	do	do	do	2.46
117	Michigan	do	Unsound	2.84
118	Ohio	do	Sound	2.88
119	do	do	Unsound	3.37
120	do	do	Sound	3.44
121	do	do	do	3.71
122	do	do	Unsound	6.95
123	do	Argillaceous dolomite	do	.84
124	do	do	do	1.25
125	do	do	do	1.26
126	Indiana	do	do	1.55
127	do	do	do	1.81
128	Arkansas	do	do	2.15
129	Indiana	do	do	2.28
130	do	do	do	2.73
131	do	do	do	2.80
132	Virginia	Siliceous dolomite	Sound	.09
133	Michigan	do	Unsound	2.24
134	Arkansas	Sandstone	Sound	.69
135	do	do	Unsound	1.13
136	do	do	Sound	1.63
137	New York	do	Unsound	1.70
138	Kentucky	do	Sound	1.81
139	Arkansas	do	do	1.93
140	Kentucky	do	do	2.00
141	Minnesota	do	Unsound	4.34
142	Arkansas	Feldspathic sandstone	Sound	.79
143	New York	do	do	1.00
144	Arkansas	do	do	1.10
145	do	do	do	1.43
146	Kentucky	do	Unsound	2.21
147	do	do	do	2.65
148	do	do	do	2.86
149	Mexico	Calcareous sandstone	Sound	.15
150	New Mexico	do	Unsound	1.15

(Continued on page 229)

INSTRUMENT DEVELOPED FOR MEASURING LENGTH OF CRACKS IN CONCRETE

Reported by H. L. BOSLEY, Assistant Materials Engineer, United States Bureau of Public Roads

AN INSTRUMENT for use in measuring the length of cracks in concrete pavements and with which measurements can be made rapidly and with a satisfactory degree of accuracy has recently been designed by the Division of Tests of the Bureau of Public Roads. The instrument was made for use on concrete surfaces in making condition surveys in connection with subgrade studies and to determine the effect of various features of design.

It is thought that a description of the instrument may be of interest to State highway organizations and other agencies who are making condition surveys. Such a

study of concrete pavements usually begins with a determination of the amount of cracking which has occurred, and if carefully done with a tape involves considerable painstaking labor.

The apparatus, which embodies the same principle as the chartometer used for measuring distances on maps, consists of a wheel which is rolled along the surface to be measured by means of an operating handle. The circumference of the wheel being known, it is so geared to a revolution counter that for each linear foot of travel one unit is recorded on the counter. Thus, the reading on the counter at any time shows the total distance over which the wheel has been rolled.

Before giving a detailed description of the device, a general view of which is shown in Figure 1, it may be well to discuss its usefulness. One man with the instrument can replace two men with a tape and obtain the same data more easily and it is believed more accurately. A second advantage is that the actual length of meandering cracks can be accurately measured, something which can not readily be done with a tape. Calibration tests with one of these devices indicated that lengths

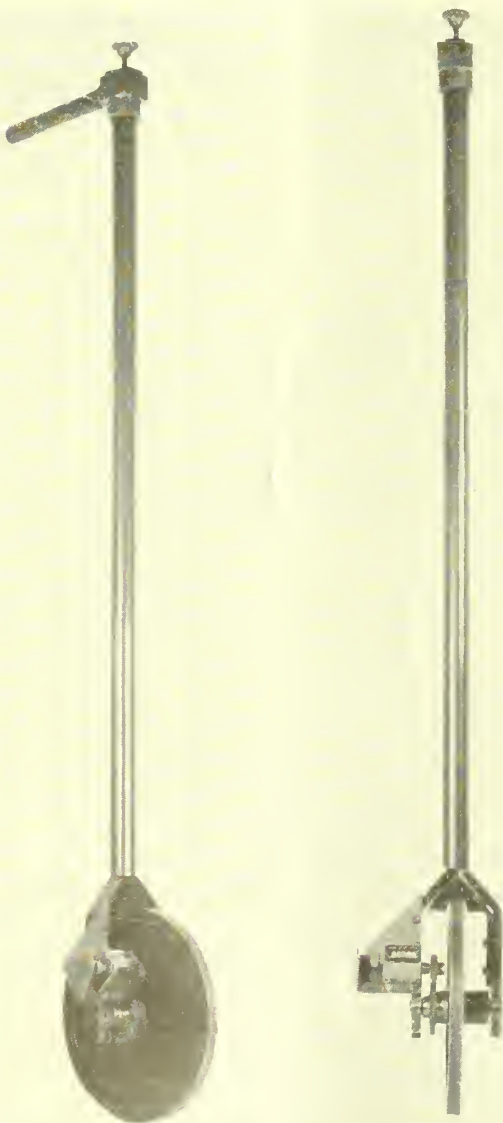


FIG. 1.—TWO VIEWS OF APPARATUS USED IN MAKING SURVEYS

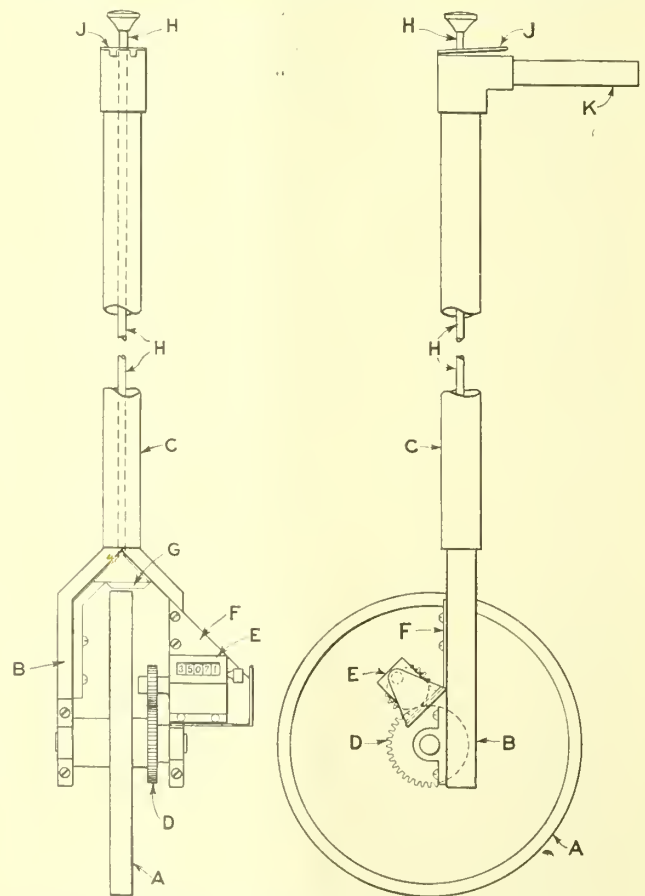


FIG. 2.—DRAWING SHOWING DETAILS OF APPARATUS

measured at the speed of a man walking were accurate to within 0.25 per cent or 1 foot in 400. A check on the individual measurements is available at all times in the total on the counter which will record nearly 20 miles before clearing.

The details of construction are shown in Figure 2. A steel wheel, A, whose circumference is exactly 2 feet, rolls freely in bearings supported by the forked frame, B, attached to the tubular handle, C. A pair of spur gears, D, rotate the spindle of the revolution counter, E, twice for each revolution of the measuring wheel, A. Thus for each distance of 1 foot traversed one unit is registered on the counter. The counter, E, is supported and protected by a guard bracket, F. In transporting the apparatus from one place to another it is desirable to prevent rotation of the counter spindle. For this purpose a rubber brake shoe, G, is provided and this is pressed against the measuring wheel, A, by the rod, H, which terminates in a knob at the end of the operating handle. The operator simply presses down on this knob and locks the wheel, the brake being held in contact by the latch, J. To release the brake a slight pressure on the end of the latch, J, frees the rod, H, and permits the wheel, A, to again roll freely.

The operation in the field is so simple and obvious that it needs no description.

(Continued from page 224)

State-aid routes. The highways classified include, therefore, all routes which carry any considerable volume of present and expected future traffic. The traffic classification for each section is shown in the full report.

The mileage so classified in each of the classification groups is shown in Table 14.

TABLE 14.—Traffic classification of Vermont highways ¹

Traffic classification	Miles	Per cent of total miles
Major 1.....	49.1	2.2
Major 2.....	129.5	5.9
Total major.....	178.6	8.1
Medium 1.....	88.9	4.1
Medium 2.....	84.1	3.8
Medium 3.....	342.7	15.6
Total medium.....	515.7	23.5
Minor 1.....	215.1	9.8
Minor 2.....	1,284.4	58.6
Total minor.....	1,499.5	68.4
Total all routes.....	2,193.8	100.0

¹ Highways classified include all routes carrying any appreciable density of traffic in 1926.

IMPROVEMENT PLAN PRESENTED

The full report presents a proposed plan of improvement, based upon the analysis of the Vermont Federal-aid highway system,⁸ considering present improvements and traffic, shows that 275.5 miles of new construction superior to gravel surfaces will be necessary to meet traffic requirements during the five-year period from January 1, 1927, to December 31, 1931.

⁸ The Vermont plan of highway improvement is limited to the Federal-aid system since less than 20 miles not included in the Federal-aid system carried sufficient traffic to justify their inclusion in the five-year plan of improvement.

The cost of improving these highways with surfaces adequate to serve present and expected future traffic as estimated by the Vermont State Highway Department will be approximately \$12,000,000. This estimate includes the cost of bridges and structures, grading, and surface costs.

By 1931, because of the normal increase in traffic, 278.5 additional miles of Federal-aid highways are expected to carry 800 or more vehicles and will require surfaces superior to gravel. To serve the traffic on these routes adequately, construction of surfaces superior to gravel on this mileage will be required between 1932 and 1936. Since the major part of this mileage will be on medium traffic routes, the total cost of improving the 278.5 miles included in this group should be considerably less than the required expenditures during the period 1927 to 1931. This decrease, however, will be partially offset by the need for improvement of a comparatively small mileage of routes, approximately 80 miles, not included in the Federal-aid system, with surfaces superior to gravel between 1926 and 1936.

It is believed that a construction program more limited in scope than the proposed five-year program will result in increased total highway expenditures, because of the higher maintenance costs which may be expected to result from postponement of adequate improvement; and that it will also result in inadequate highway service and increased motor-vehicle operating costs.

For the most economical accomplishment of the proposed improvement plan it is recommended that substantially the present Federal-aid highway system be established as a primary or State highway system under the jurisdiction of the State highway department as to construction, maintenance, and control.

It is also recommended that a secondary highway system be established. This system should include the more important traffic routes not included in the primary system.

The establishment of these systems and their improvement in accordance with the foregoing plan will insure a systematic and scientific development of Vermont State highways.

(Continued from page 227)

TABLE 2.—Geographical distribution of test samples

State or country	Lime-stone	Dolomite	Sand-stone	Marble	Total
Alabama.....	1				1
Arkansas.....		1	7		8
Florida.....	4				4
Illinois.....	4	1			5
Indiana.....	1	7			8
Kansas.....	8				8
Kentucky.....			5		5
Louisiana.....	1			1	2
Maryland.....	5			1	6
Michigan.....		3			3
Minnesota.....	1		1		2
Missouri.....	1				1
New Mexico.....			1		1
New York.....	8	3	2	1	14
Ohio.....	10	36			46
Oklahoma.....	3				3
Pennsylvania.....	2	1			3
Rhode Island.....	1				1
South Dakota.....	1	1			2
Tennessee.....	2				2
Texas.....	17			1	18
Virginia.....		2			2
West Virginia.....	2		1		3
Wyoming.....		1			1
Mexico.....			1		1

YADKIN RIVER BRIDGE TEST COMPLETED

Tests of the reinforced concrete arch bridge over the Yadkin River between Albemarle and Mount Gilead, N. C.—involving the structure as it stood in service and with the continuity of the superstructure destroyed—were completed early in December.

The purpose of the stress measurements was to determine the manner in which a full-sized concrete arch, built under normal conditions, deforms under determinate loads, and to compare these deformations and the stresses produced in the arch rib with the deformations and stresses as computed by the generally accepted elastic theory of arches and those derived from the tests of models by means of Beggs deformeter gauges.

Loads have been imposed by means of tanks of water placed on the span. These tanks are made of wood,



YADKIN RIVER BRIDGE TO BE TESTED TO DESTRUCTION

12 by 20 by 18 feet inside dimensions, and weigh, when full, 160 tons each.

Data has been secured with the following conditions of loading:

1. A complete series of loads on the bridge as it stood originally; using one loading tank.

2. A complete series of loads on the bridge with the continuity of the superstructure destroyed, so as to approach as near as possible to the conditions generally assumed in designing arches; using one loading tank.

3. With the continuity of the bridge destroyed, loading it in the most critical positions, with two tanks.

The tanks were weighed by means of the deformation of supporting copper cylinders set in steel cylinders and receiving the load from closely fitting pistons. Specially annealed copper cylinders (one-half inch by one-half inch) accurately shaped were placed in the cells so that they carry the entire weight of the tank placed on the pistons. The weight, a function of the deformation of the copper cylinders, was taken from calibration curves previously determined in the laboratory. By this means the empty-tank weight of 47,000 pounds was checked within 300 pounds of the computed weight and other check measurements.

The following measurements were made with various positions of load and three increments of water for each position:

1. The deformation of the concrete and steel at 24 points on the arch rib. From these measurements the stress is determined by the use of the modulus of elasticity derived from tests of specimens taken from the bridge.

The deformation was measured by electric telemeters which are designed on the principle that the electrical resistance of a carbon pile varies with the pressure on the pile. The instrument was attached to steel plugs grouted in the concrete arch rib so that the deformation movements are transmitted as pressures upon the carbon piles. The telemeters are read to one ten-thousandth of an inch and estimated to one hundred-thousandth of an inch.

2. The angular rotation of the arch ring at nine points.

This measurement was made by means of a clinometer, consisting of a steel bar carrying a level bubble and leveling screw attached to a dial reading to one-thousandth of an inch. The instrument was placed and leveled on gauge points located 20 inches apart on steel plugs in the arch rib. The relative displacement of the gauge points was measured, and from these measurements the angular rotation under any increment of load is derived directly.

3. The change in length of middle ordinates of 5-foot arcs of the axis at 5-foot intervals over the entire length of the arch rib.

The middle-ordinate changes were measured by means of a radius meter. This instrument consists of a steel bar 5 feet long, carrying at its center a Federal dial reading to one ten-thousandth of an inch. Gauge points at the ends of the radius meter are placed on corresponding points on steel plugs in the arch and the stem of the dial was allowed to rest on the surface of a square steel plug halfway between the gauge points. The movements of the square plug are transferred to the needle and read on the dial.

4. Deflections at each spandrel point of the arch.

The deflections were measured from a wire suspended over a pulley with a counterweight at one end to maintain a constant tension and sag. Points were located in the concrete at the top of each spandrel column and the deflections measured from these points to the wire.

5. Rotation of the piers at the end of the span under observation.

The pier rotations were measured by clinometers in the same manner as the axial rotations, the gauge points being placed on the top of each pier. These rotation measurements were checked by means of two wires placed 5 feet apart vertically and suspended independently of the span under observation. Movements were shown by the relative longitudinal displacements of fixed points on the wires and on the ends of the piers. Although the field work is now complete it will be several months before the data can be completely analyzed and conclusions reached.

ROAD PUBLICATIONS OF BUREAU OF PUBLIC ROADS

Applicants are urgently requested to ask only for those publications in which they are particularly interested. The Department can not undertake to supply complete sets nor to send free more than one copy of any publication to any one person. The editions of some of the publications are necessarily limited, and when the Department's free supply is exhausted and no funds are available for procuring additional copies, applicants are referred to the Superintendent of Documents, Government Printing Office, this city, who has them for sale at a nominal price, under the law of January 12, 1895. Those publications in this list, the Department supply of which is exhausted, can only be secured by purchase from the Superintendent of Documents, who is not authorized to furnish publications free.

ANNUAL REPORTS

Report of the Chief of the Bureau of Public Roads, 1924.
Report of the Chief of the Bureau of Public Roads, 1925.
Report of the Chief of the Bureau of Public Roads, 1927.

DEPARTMENT BULLETINS

- No. 105D. Progress Report of Experiments in Dust Prevention and Road Preservation, 1913.
*136D. Highway Bonds. 20c.
220D. Road Models.
257D. Progress Report of Experiments in Dust Prevention and Road Preservation, 1914.
*314D. Methods for the Examination of Bituminous Road Materials. 10c.
*347D. Methods for the Determination of the Physical Properties of Road-Building Rock. 10c.
*370D. The Results of Physical Tests of Road-Building Rock. 15c.
386D. Public Road Mileage and Revenues in the Middle Atlantic States, 1914.
387D. Public Road Mileage and Revenues in the Southern States, 1914.
388D. Public Road Mileage and Revenues in the New England States, 1914.
390D. Public Road Mileage and Revenues in the United States, 1914. A Summary.
407D. Progress Reports of Experiments in Dust Prevention and Road Preservation, 1915.
*463D. Earth, sand-clay and gravel. 15c.
*532D. The Expansion and Contraction of Concrete and Concrete Roads. 10c.
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*660D. Highway Cost Keeping. 10c.
*670D. The Results of Physical Tests of Road-Building Rock in 1916 and 1917. 5c.
*691D. Typical Specifications for Bituminous Road Materials. 10c.
*724D. Drainage Methods and Foundations for County Roads. 20c.
*1077D. Portland Cement Concrete Roads. 15c.

DEPARTMENT BULLETINS—Continued

- No. *1132D. The Results of Physical Tests of Road-Building Rock from 1916 to 1921, Inclusive. 10c.
1259D. Standard Specifications for Steel Highway Bridges, adopted by the American Association of State Highway Officials and approved by the Secretary of Agriculture for use in connection with Federal-aid road work.
1279D. Rural Highway Mileage, Income, and Expenditures, 1921 and 1922.
1486D. Highway Bridge Location.

DEPARTMENT CIRCULARS

- No. 94C. T. N. T. as a Blasting Explosive.
331C. Standard Specifications for Corrugated Metal Pipe Culverts.

MISCELLANEOUS CIRCULARS

- No. 62M. Standards Governing Plans, Specifications, Contract Forms, and Estimates for Federal Aid Highway Projects.
93M. Direct Production Costs of Broken Stone.
105M. Federal Legislation Providing for Federal Aid in Highway Construction and the Construction of National Forest Roads and Trails.

FARMERS' BULLETINS

- No. *338F. Macadam Roads. 5c.
*505F. Benefits of Improved Roads. 5c.

SEPARATE REPRINTS FROM THE YEARBOOK

- No. *739Y. Federal Aid to Highways, 1917. 5c.
*849Y. Roads. 5c.
914Y. Highways and Highway Transportation.

REPRINTS FROM THE JOURNAL OF AGRICULTURAL RESEARCH

- Vol. 5, No. 17, D- 2. Effect of Controllable Variables upon the Penetration Test for Asphalts and Asphalt Cements.
Vol. 5, No. 19, D- 3. Relation Between Properties of Hardness and Toughness of Road-Building Rock.
Vol. 5, No. 24, D- 6. A New Penetration Needle for Use in Testing Bituminous Materials.
Vol. 6, No. 6, D- 8. Tests of Three Large-Sized Reinforced-Concrete Slabs Under Concentrated Loading.
Vol. 11, No. 10, D-15. Tests of a Large-Sized Reinforced-Concrete Slab Subjected to Eccentric Concentrated Loads.

*Department supply exhausted.

UNITED STATES DEPARTMENT OF AGRICULTURE
BUREAU OF PUBLIC ROADS

STATUS OF FEDERAL AID HIGHWAY CONSTRUCTION

AS OF

NOVEMBER 30, 1927

FISCAL YEARS 1917-1927

FISCAL YEAR 1928

STATES	FISCAL YEARS 1917-1927				FISCAL YEAR 1928				BALANCE OF FEDERAL AID FUND AVAILABLE FOR NEW PROJECTS	STATES								
	PROJECTS COMPLETED PRIOR TO JULY 1, 1927				PROJECTS COMPLETED SINCE JUNE 30, 1927						* PROJECTS UNDER CONSTRUCTION				PROJECTS APPROVED FOR CONSTRUCTION			
	TOTAL COST	FEDERAL AID	MILES		TOTAL COST	FEDERAL AID	MILES				ESTIMATED COST	FEDERAL AID ALLOTTED	MILES		ESTIMATED COST	FEDERAL AID ALLOTTED	MILES	
Alabama	\$ 20,051,371.68	\$ 9,615,099.94	1,400.2		\$ 126,805.90	\$ 61,502.01	1.9		\$ 8,300,874.90	\$ 3,995,072.54	457.7		\$ 290,282.96	\$ 145,141.47	26.9		\$ 2,079,779.04	Alabama
Arizona	11,809,950.70	6,447,169.27	800.8		267,759.35	192,918.61	14.1		1,448,055.72	1,086,686.44	67.8		34,528.88	17,264.44	0.3		2,928,204.24	Arizona
Arkansas	22,537,014.63	9,595,192.75	1,560.6		52,245.29	25,122.64	0.1		3,873,634.24	1,881,911.81	223.9		753,929.34	177,021.26	7.7		1,273,751.54	Arkansas
California	35,128,269.04	16,967,065.82	1,306.3		2,553,427.98	1,126,574.13	63.0		7,505,092.02	3,461,589.86	156.8		251,941.50	147,766.33	9.5		2,853,294.86	California
Colorado	15,497,121.91	7,934,298.91	829.0		91,935.74	48,486.64	0.9		6,545,381.81	3,243,759.50	285.1		3,243,759.50	20,057.34	2.9		2,448,719.61	Colorado
Connecticut	6,397,392.29	2,444,000.54	137.3		1,324,934.52	355,348.84	17.7		5,737,303.87	1,609,650.87	69.6		484,816.75	82,330.00	5.5		315,045.75	Connecticut
Delaware	6,337,026.55	2,345,572.42	159.5		633,662.17	120,820.22	14.6		998,398.73	365,371.18	31.2		777,845.42	269,730.00	18.0		7,919.18	Delaware
Florida	7,476,856.31	3,627,912.60	245.1		1,517,818.43	734,206.79	32.7		8,106,464.00	3,545,343.34	176.3						807,212.27	Florida
Georgia	31,951,435.50	15,101,232.40	2,173.6		4,589,628.78	2,170,968.68	209.2		6,027,113.97	2,937,945.54	182.4						201,015.38	Georgia
Idaho	13,225,515.45	7,075,527.16	835.5		973,835.68	633,475.61	91.3		2,130,996.90	1,281,122.05	131.8		654,939.54	397,473.70	40.8		107,221.48	Idaho
Illinois	48,538,982.16	22,781,516.60	1,530.8		408,494.92	191,977.32	15.0		14,907,964.22	7,099,766.97	504.9		2,573,252.62	1,257,728.01	89.7		1,655,537.50	Illinois
Indiana	23,372,717.74	11,239,559.20	732.5		439,555.13	211,985.95	13.3		17,491,140.39	8,366,213.54	534.0		469,700.00	39,800.00	17.0		254,558.31	Indiana
Iowa	34,305,138.86	14,395,603.75	2,484.4		1,739,474.81	845,094.32	168.2		14,099,057.85	6,075,463.78	447.5		745,407.49	108,923.66	5.8		104,446.49	Iowa
Kansas	37,442,061.61	14,730,823.48	1,495.2		1,211,448.88	559,497.96	57.3		1,768,306.07	819,577.68	737.2		307,934.37	14,704.30	53.1		146,375.12	Kansas
Kentucky	23,216,500.53	9,910,694.75	874.9		683,973.69	323,644.21	32.3		10,693,439.90	4,731,977.34	439.8		29,398.61	14,704.30	13.9		19,735.40	Kentucky
Louisiana	15,877,552.20	7,093,692.21	1,178.7		931,554.63	442,316.21	37.9		4,075,981.89	1,850,920.80	147.1		1,357,070.56	683,870.27	29.0		214,716.51	Louisiana
Maine	10,564,800.06	4,858,452.67	357.6		1,098,204.92	443,485.16	40.5		1,886,415.44	593,027.40	141.0		399,073.01	156,774.91	15.9		1,053,981.82	Maine
Maryland	11,790,203.93	5,524,938.27	477.8						2,245,508.14	1,025,285.64	115.4						9,982.09	Maryland
Massachusetts	20,570,246.02	7,425,928.15	410.4		773,728.93	159,264.74	9.6		6,742,463.44	1,955,290.01	114.5		978,644.10	354,117.00	25.4		1,748,343.10	Massachusetts
Michigan	31,977,248.37	14,328,484.99	1,084.2		1,236,846.69	572,143.94	57.7		15,217,455.17	6,914,699.93	400.6						477,610.14	Michigan
Minnesota	45,099,548.47	19,045,148.57	3,043.5		3,933,017.05	1,412,498.90	155.9		4,890,575.77	1,135,000.00	232.5		331,800.98	6,000.00	13.7		112,676.43	Minnesota
Mississippi	19,331,230.75	9,004,294.62	1,314.1		1,220,054.38	604,408.40	74.5		6,462,539.46	3,132,245.45	337.9		238,738.19	119,369.09	18.5		575,579.44	Mississippi
Missouri	42,389,290.41	19,681,056.48	1,944.8		2,690,146.75	1,114,656.86	89.3		9,434,609.07	3,691,765.26	273.8		112,891.83	56,445.91	6.7		647,706.49	Missouri
Montana	12,694,955.72	7,287,288.69	1,151.6		371,952.96	285,259.38	50.5		3,701,995.68	2,557,065.08	283.6		1,378,789.51	766,162.03	151.1		4,050,608.82	Montana
Nebraska	16,157,040.26	7,739,386.39	2,246.6		2,552,969.23	1,229,026.64	243.6		13,297,972.92	6,551,358.91	1,314.2		639,568.27	318,530.18	64.0		382,070.88	Nebraska
Nevada	10,421,349.31	7,589,188.58	853.6		321,310.62	279,255.42	52.9		1,902,033.36	1,652,888.91	186.1		105,947.54	92,397.17	14.9		129,464.82	Nevada
New Hampshire	5,668,997.76	2,778,928.05	284.9		211,962.88	101,807.91	7.0		1,434,838.85	623,139.51	42.3		70,803.54	28,110.00	1.9		3,131.53	New Hampshire
New Jersey	22,228,240.08	7,495,354.48	316.3		2,467,874.39	593,910.00	39.6		5,821,652.21	1,312,753.56	85.4						2.96	New Jersey
New Mexico	13,336,250.94	7,937,598.06	1,505.2		551,280.81	341,073.59	45.5		3,075,115.33	2,417,463.28	219.7		45,267.58	28,713.22	2.3		1,434,300.85	New Mexico
New York	54,183,095.44	21,693,955.65	1,439.3		2,833,004.65	909,807.94	61.8		44,047,036.00	10,759,628.95	665.9		5,522,600.00	1,133,302.50	73.3		3,187,717.06	New York
North Carolina	35,295,849.21	14,518,903.15	1,450.1		1,568,316.60	729,783.35	63.3		3,295,184.21	1,570,015.52	92.0		34,105.00	16,500.00	0.9		595,359.97	North Carolina
North Dakota	15,881,559.55	7,746,293.68	2,715.6		2,510,498.95	1,457,967.40	280.4		4,955,518.78	2,392,951.09	755.4		474,665.97	221,565.82	111.9		125,041.01	North Dakota
Ohio	52,621,391.49	19,331,376.76	1,515.0		1,585,696.05	694,089.42	51.5		13,135,144.28	5,116,214.45	352.4		2,980,302.00	1,157,285.00	66.5		2,195,053.57	Ohio
Oklahoma	30,381,957.08	14,117,589.21	1,268.1		789,742.25	379,754.99	9.6		4,481,386.92	1,898,633.12	309.8		2,755,484.18	1,395,553.64	154.1		20,147.04	Oklahoma
Oregon	13,583,584.76	10,411,452.94	1,065.0		224,521.65	121,684.57	1.8		2,500,108.61	1,302,136.77	74.1		1,202,502.52	72,800.41	3.2		523,474.31	Oregon
Pennsylvania	77,226,174.22	31,677,620.32	1,534.3		2,693,226.73	800,092.95	59.5		19,513,433.28	9,911,319.10	370.3		1,907,799.38	597,383.33	39.6		1,048,100.30	Pennsylvania
Rhode Island	5,233,413.38	1,998,479.06	115.0		700,482.52	227,205.00	15.1		970,453.99	353,163.74	16.0		835,421.77	223,783.57	13.9		330,562.53	Rhode Island
South Carolina	17,002,039.93	7,526,985.80	1,568.4		1,696,916.12	808,011.50	41.7		8,425,077.75	2,579,544.54	253.5		292,188.76	23,995.94	0.4		117,967.72	South Carolina
South Dakota	15,262,063.24	9,507,555.82	2,502.9		277,881.18	149,826.87	56.1		4,846,875.83	2,560,847.52	712.7		364,424.34	137,906.07	55.5		31,049.00	South Dakota
Tennessee	24,283,035.03	11,551,457.55	868.7		1,654,819.20	734,304.01	31.7		9,448,066.70	3,935,524.01	249.3		378,552.21	189,276.10	13.0		483,795.33	Tennessee
Texas	78,130,235.07	31,566,950.45	4,665.4		6,393,551.28	2,876,010.13	241.1		12,427,619.15	5,562,910.05	373.1		4,895,702.72	1,989,532.05	192.7		3,108,981.32	Texas
Utah	9,154,377.33	3,749,855.01	629.9		528,149.58	398,951.41	46.2		2,775,902.77	2,060,667.58	130.1		510,355.98	250,706.94	21.1		199,379.12	Utah
Vermont	5,037,119.23	2,348,855.01	152.7		1,028,958.37	530,343.92	26.5		2,072,322.11	724,824.03	45.0						30,108.04	Vermont
Virginia	26,844,025.24	12,537,143.25	1,188.9		455,975.42	202,124.28	1.0		1,824,000.00	2,174,341.34	116.3		26,248.75	513,000.00	0.2		271,080.90	Virginia
Washington	18,184,505.97	8,246,551.95	711.1		847,151.95	415,736.32	5.3		2,918,690.33	1,310,600.00	66.5		1,563,789.22	513,000.00	61.2		771,419.73	Washington
West Virginia	10,424,947.32	4,573,748.01	419.4		2,794,079.46	1,371,637.81	134.8		8,341,538.35	3,431,427.28	278.0		282,892.22	137,623.76	9.0		3,347.95	West Virginia
Wisconsin	12,891,502.16	7,187,658.90	1,769.5		230,270.51	149,562.10	24.2		9,436,982.12	4,188,900.97	326.3		105,044.96	67,439.84	24.9		1,900,852.32	Wisconsin
Wyoming	12,537,267.05	7,187,658.90	1,315.3		333,553.15	149,562.10	24.2		3,171,669.23	2,028,733.03	288.0						115,641.36	Wyoming
Hawaii	343,584.15		6.5		160,636.91	70,440.00	6.7										905,976.36	Hawaii
TOTALS	1,154,740,601.48	510,007,691.24	60,957.5		63,889,413.50	28,197,766.34	2,735.5		354,855,098.35	150,820,606.85	13,899.9		35,725,653.18	13,785,713.96	1,455.9		41,689,421.61	TOTALS

* Includes projects reported completed (final vouchers not yet paid) totaling: Estimated cost: \$ 55,625,438.59 Miles: 5,021.1

* Includes projects reported completed (final vouchers not yet paid) totaling: Estimated cost \$ 130,266,567.21 Federal aid \$ 55,625,438.59 Miles 5,021.1

PUBLIC ROADS

A JOURNAL OF HIGHWAY RESEARCH



UNITED STATES DEPARTMENT OF AGRICULTURE
BUREAU OF PUBLIC ROADS



VOL. 8, NO. 11



JANUARY, 1928



THE PROBLEM PRESENTED BY RAILROAD GRADE CROSSINGS IS A SERIOUS ONE

PUBLIC ROADS

A JOURNAL OF HIGHWAY RESEARCH

U. S. DEPARTMENT OF AGRICULTURE

BUREAU OF PUBLIC ROADS

CERTIFICATE: By direction of the Secretary of Agriculture, the matter contained herein is published as administrative information and is required for the proper transaction of the public business

The reports of research published in this magazine are necessarily qualified by the conditions of the tests from which the data are obtained. Whenever it is deemed possible to do so, generalizations are drawn from the results of the tests; and, unless this is done the conclusions formulated must be considered as specifically pertinent only to the described conditions

VOL. 8, NO. 11

JANUARY, 1928

R. E. ROYALL, Editor

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Owing to the necessarily limited edition of this publication it will be impossible to distribute it free to any persons or institutions other than State and county officials actually engaged in planning or constructing public highways, instructors in highway engineering, periodicals upon an exchange basis, and Members of both Houses of Congress. At the present time names can be added to the free list only as vacancies occur. Others desiring to obtain "Public Roads" can do so by sending 10 cents for a single number or \$1 per year to the Superintendent of Documents, U. S. Government Printing Office, Washington, D. C.

A STATISTICAL ANALYSIS OF HIGHWAY-RAILROAD GRADE-CROSSING ACCIDENTS IN 1926

Reported by A. B. FLETCHER, Consulting Highway Engineer, and WILLIAM G. ELIOT, 3d, Associate Highway Economist, United States Bureau of Public Roads

THE PROBLEM of the highway-railroad grade crossing, serious enough when the highways carried nothing faster than horse-drawn vehicles, and not very many of them, has become much more urgent since the advent of high-speed automotive traffic. Figures published by the Bureau of Statistics of the Interstate Commerce Commission show a rapid increase in grade-crossing fatalities in the United States, especially during the past 10 years. From a total of 1,652 deaths at steam railroad grade crossings in 1916 the annual toll increased to 2,491 in 1926.¹ While this increase is by no means in proportion to the increased use of the highways, grade crossings, nevertheless, remain one of the most important hazards to traffic.

CONCLUSIONS SUMMARIZED

In the belief that the most intelligent program for grade-crossing safety depends upon a thorough knowledge of the existing conditions, the United States Bureau of Public Roads has recently completed a careful analysis of nearly 6,000 grade-crossing accidents as reported to the Interstate Commerce Commission by the steam railroads of the country during 1926. These reports are public records and are open to examination without special arrangement with the commission, though it is only fitting to state that the officials of its Bureau of Statistics evinced great interest in the work and gave special facilities for the study.

Some of the more definite conclusions to which this study points may be summarized as follows:

There is a real need for a series of intensive studies of individual grade-crossing accidents, as well as for more complete and accurate reporting of grade-crossing accident data in general.

The hazard at grade crossings in rural areas is far more important than has heretofore been reported or understood. At least 16 per cent of all fatalities due to accidents on the rural highways of the country are attributable to the crossings of the railroads and highways at grade.

A serious effort should be made to hasten the change of thousands of crossings from the "unprotected" to the "protected" class by the installation of suitable warning devices and continued attention given to the improvement of existing protection.

Despite all external safeguards, it will be necessary by persistent and forceful publicity to keep in the minds of all highway users a realization of the hazards to which they are exposed as long as any considerable proportion of the 235,000 steam railroad grade crossings in the United States continue to exist.

THE SOURCE OF DATA

Under the act of Congress of May 6, 1910, each railroad is required to report monthly to the Interstate Commerce Commission all accidents "arising from the operation of such railroad under such rules and regulations as may be prescribed by the said commission."

Under the rules of the commission effective January 1, 1922,—

a reportable accident is an accident arising from the operation of a railway that results in one or more of the following circumstances:

(a) Damage to railway property amounting to more than \$150, including the expense of clearing wreck * * *.

(b) Death of a person.

(c) Injury to an employee sufficient to incapacitate him from performing his ordinary duties for more than three days in the aggregate during the 10 days immediately following the accident. This rule * * * does not apply to employees classed as passengers or trespassers.

(d) Injury to a person other than an employee if the injury is sufficient, in the opinion of the reporting officer, to incapacitate the injured person from following his customary vocation or mode of life for a period of more than one day * * *.



THE RESULT OF A GRADE-CROSSING ACCIDENT

In general, the reports cover—

accidents at public highway grade crossings due to trains, locomotives, or cars striking pedestrians or colliding with trolley cars, automobiles, or other vehicles, or objects other than hand cars or other railway tools or material * * *.

In the classification of accidents by the commission this has been extended to include such miscellaneous cases as injuries to crossing flagmen when struck by highway vehicles, injuries to persons who have jumped from vehicles in anticipation of collisions which do not occur, and injuries resulting from the wrecking of highway vehicles in averting collisions with trains, when such accidents occur on the railroad right of way.

Under the rules of the commission, "Any person killed in an accident at the time of its occurrence, or so seriously injured as to die within 24 hours thereafter, should be reported as killed." All other personal casualties are listed as injured, even though death follows after a period of more than 24 hours. No attempt was made in this study to tabulate these later deaths, but the Interstate Commerce Commission, as a result of a recent special analysis,² reports that in addition to its published totals of deaths, there was a total of 205 of these subsequent "fatalities" from grade-crossing accidents in the entire country during

¹ See Accident Bulletin No. 94, Interstate Commerce Commission, p. 113.

² Made for this study at the request of the Bureau of Public Roads

1926. This amounts to an increase of 8 per cent over the commonly accepted and quoted figures. In the following study, however, the killed and injured are listed as reported by the railroads, without allowance for subsequent fatalities.

The railroads are instructed to—

state all material facts, including whether injured persons were pedestrians, occupants of trolley cars, automobiles, or other vehicles, riding bicycles or motor cycles, etc.; the nature of protection, if any, afforded at the crossing at the time of an accident; that is, whether crossing gates, watchman, audible or visual signals (not stationary signs), etc.; the kind of vehicle or object struck, and whether the view of crossing was obscured by buildings, trees, cars, or other objects.

GREAT VARIETY OF ACCIDENTS REPORTED

The reporting officers of the railroads differ widely in their interpretation of the phrase "material facts," and often fail to supply some of the details specifically enumerated above. This may be partly due to the fear of encouraging litigation, despite the legal provision that no accident report "shall be admitted as evidence or used for any purpose in any suit or action for damages growing out of any matter mentioned in said report." Many of the reports are merely perfunctory, but despite these limitations there is no comparable source for nation-wide grade-crossing accident facts.

The Interstate Commerce Commission, through its Bureau of Statistics, has tabulated a considerable volume of data from these reports, including the casualties at grade crossings and whether the victims were trespassers at the time; the type of accident, whether involving pedestrian, passenger automobile, truck, etc.; and the type of protection afforded at the crossings for each type of accident. These appear in quarterly reports and in the annual Accident Bulletin published by the commission. Certain of the analyses presented here are practically duplications of those previously made by the commission. For the most part, however, the study deals with what are believed to be new and hitherto unpublished data.

The method followed in the present analysis included copying off of pertinent data, codifying these data wherever that had not already been done by the Interstate Commerce Commission, transferring them to punch cards, and finally tabulating them by machine. No attempt was made to secure accurate correspondence of final results with the figures published by the commission. The total of 5,808 accidents analyzed in this study falls short of the 5,890 tabulated by the Interstate Commerce Commission. The deficiency is principally accounted for by the fact that some of the reports were misclassified by the railroads, were not in their proper place in the files, and escaped the present tabulation. On the other hand, a few of the accidents included in the commission's totals were found to be erroneously classified and were deliberately discarded. Various other differing judgments or minor errors in both tabulations make strict comparability impossible either as to number of accidents or as to details of those tabulated.

The greatest conceivable variety of grade-crossing mishaps was found in these reports. Interspersed in the monotonous sequence of pedestrians, automobiles, and teams blundering on to crossings and being struck by trains that could not turn aside nor stop quickly were frequent unique combinations of circumstances and, occasionally, a note of near-comedy. In one case

a helpful neighbor and several members of his family met death when the friend whose car they were towing applied his brakes suddenly and stalled the first car in the path of a fast train. In another case, it was an excited woman passenger who grabbed the emergency brake and stalled the automobile on the track. One driver, after stopping on the track, backed off safely but continued backing in a semicircle until he was once more in front of the train.

A pedestrian standing close to the track was knocked down and injured when struck by the body of a switchman who was riding the side of a car. In yet another case a passenger on a train was thrown down and hurt when the engineman applied the emergency brakes as a truck ran into the side of his engine. An unlucky farmer lost his life while "endeavoring to urge cow over crossing." An intoxicated operator drove his car into the rear of another car waiting at the crossing and pushed it in front of an approaching train. Numerous cases were reported in which bystanders were struck by flying wreckage, sometimes with fatal results. Several persons lost their lives in futile attempts to push their stalled cars off the tracks.

RURAL AND URBAN ACCIDENTS COMPARED

One of the major questions in the minds of those responsible for this study was as to the distribution of accidents between rural and urban regions. It was believed that grade-crossing accidents would be found to constitute a much greater proportion of all traffic accidents on the country highways than on city streets, due to the differing traffic conditions. This conclusion was based on the assumptions, (1) that grade crossings in the cities are either eliminated or are better protected than in the country, while both rail and highway vehicles are moving at slower speeds; and (2) that because of the congestion of vehicles and pedestrians on city streets, a majority of all traffic accidents occur in the cities and larger villages.

According to the most recent estimate made by the National Safety Council, there were in 1926 approximately 25,000 fatalities due to street and highway accidents of all kinds. Railroad grade-crossing accidents, according to the Interstate Commerce Commission figures, account for 2,491 of these, or 10 per cent. Analysis of these statistics to show urban and rural accidents separately involves further estimates, which can be made, it is believed, with sufficient accuracy to be worth while.

The proportion of urban and rural grade-crossing accidents was determined from an analysis of the individual grade-crossing accident reports. According to the name of the "nearest station" as reported by the railroad and the "distance and direction from station named," with occasional assistance from other data given, the accidents were divided into two groups: (1), "Urban" occurring in incorporated places having a population (census of 1920) of 2,500 persons or more; and (2) "rural," all others. This classification is in accord with the definitions of urban and rural population as made by the Bureau of the Census. It also corresponds approximately with the provisions of Federal-aid legislation, which states that Federal aid shall not be extended to highways within incorporated places having a population of 2,500 or more, except on "that portion of any such highway or street along which, within a distance of 1 mile, the houses average more than 200 feet apart."

It was not always possible to determine whether an accident happened outside or within the corporate limits of the place named, even when the distance from the station was stated. When the distance was not given, as was frequently the case, it was necessary to assume the location as at the station or very near it. It is probable that a considerable number of rural accidents were tabulated as urban because the nearest station was urban. It is recognized that for certain purposes it would be much more helpful to know whether an accident occurred under rural conditions (light traffic, absence of buildings adjacent to the track, etc.) or under urban conditions, than to know where it occurred in relation to a certain boundary line. But such a refinement was clearly impossible without a great deal of unsatisfactory correspondence or a personal visit to the site of almost every accident, and would have raised difficult questions of definition.

The result of the rural and urban classification was not quite according to expectations. The total number of accidents was almost exactly equally divided, there being 2,897 rural accidents and 2,907 urban accidents.³ On the other hand, the rural accidents were clearly of a more serious nature. They resulted in 1,570 fatalities while urban accidents caused but 895. The number of persons injured shows less discrepancy. These figures are tabulated by States in Table 1.

GRADE CROSSINGS RESPONSIBLE FOR 16 PER CENT OF RURAL HIGHWAY FATALITIES

Statistics for other types of street and highway accidents are scarce, but such evidence as we have seems to support the assumption that the majority of such accidents take place in urban regions. The Massachusetts motor vehicle department reports that out of 669 fatal motor-vehicle accidents during the fiscal year 1926, 394, or nearly 60 per cent, occurred in business districts or thickly settled residential districts. The New York State bureau of motor vehicles reports, for the months of June and July, 1927, but 1,440 motor-vehicle accidents, with 130 deaths, in localities of less than 2,500 population as compared with 12,937 accidents and 314 deaths in places of more than 2,500 persons. This places more than 70 per cent of these fatalities in urban areas.

It is possible that these highly urbanized States do not give a fair picture of the distribution of accidents in the country as a whole, but comparable statistics for the more rural States are not available. We do have, however, certain figures for the United States registration area,⁴ published by the Bureau of the Census. According to this source, there were in 1925 (the latest figures available) a total of 19,335 motor-vehicle fatalities (including automobile collisions with street cars and railroad trains) in the registration area, of which 11,781, or 61 per cent, occurred in cities of over 10,000 population. The corresponding ratio for 1924 (not, however, including collisions with heavier vehicles) was nearly 63 per cent. Unfortunately for the complete usefulness of these authoritative statistics, they indicate only the place where death occurred, regardless of the place where the fatal injuries may have been re-

ceived. It frequently happens that injured persons are brought to city hospitals after receiving injuries at points outside of the city—so frequently that the Bureau of the Census has felt it necessary to recognize the fact in its automobile accident-mortality reports for the larger cities, listing separately, so far as obtainable, the



A CROSSING PROTECTED BY LOWERED GATES

TABLE 1.—Rural and urban grade-crossing accidents and personal casualties by States, 1926

State	Total ¹			Rural			Urban		
	Number of accidents	Persons killed	Persons injured	Number of accidents	Persons killed	Persons injured	Number of accidents	Persons killed	Persons injured
Ohio.....	541	260	600	220	130	227	320	130	372
Illinois.....	516	237	570	221	136	233	294	101	336
New York.....	412	204	448	210	115	231	202	89	217
Indiana.....	401	173	457	173	107	184	228	66	273
Pennsylvania.....	398	160	484	214	106	261	184	54	223
Michigan.....	350	133	425	141	69	157	209	64	268
California.....	280	125	301	130	82	139	150	43	162
Texas.....	238	84	313	117	61	132	121	23	181
Wisconsin.....	170	88	217	80	49	113	90	39	104
New Jersey.....	164	81	173	93	54	100	71	27	73
Florida.....	156	69	186	95	57	109	60	12	76
Iowa.....	141	54	154	73	36	80	68	18	74
Alabama.....	136	41	196	68	24	101	68	17	98
Kansas.....	131	44	155	68	33	93	63	11	103
Oklahoma.....	128	50	163	58	37	61	70	19	94
Georgia.....	127	50	163	55	30	61	72	20	102
Missouri.....	126	65	128	68	42	63	58	23	65
North Carolina.....	125	63	125	68	37	60	57	26	65
Minnesota.....	120	47	159	75	36	95	45	11	72
Tennessee.....	107	43	119	48	23	47	59	20	72
Arkansas.....	102	24	174	40	17	51	62	7	53
Mississippi.....	95	23	109	51	16	55	43	7	39
Louisiana.....	88	27	101	62	25	62	26	2	30
Washington.....	74	25	94	35	13	44	39	12	50
Colorado.....	64	26	76	39	21	39	25	5	37
South Carolina.....	62	25	83	26	15	32	36	10	51
Virginia.....	56	15	73	36	11	42	20	4	31
Nebraska.....	56	23	95	42	18	63	14	5	32
Maryland.....	55	22	71	36	20	45	19	2	26
Massachusetts.....	50	24	57	31	20	36	19	4	21
West Virginia.....	50	17	58	27	12	27	23	5	31
North Dakota.....	46	22	56	33	20	35	13	2	21
Oregon.....	30	11	32	23	10	24	7	1	8
Vermont.....	29	15	28	15	11	15	14	4	13
Connecticut.....	25	11	35	21	10	32	4	1	3
Montana.....	22	13	29	12	8	24	10	5	5
Delaware.....	22	7	27	15	7	12	7	0	15
South Dakota.....	19	7	20	16	6	16	3	1	4
Maine.....	15	21	16	11	20	12	4	1	4
Arizona.....	15	3	15	14	3	14	1	0	1
Utah.....	14	5	14	6	4	5	8	1	9
Idaho.....	14	7	25	9	7	15	5	0	10
New Hampshire.....	11	4	10	6	4	5	5	0	5
New Mexico.....	11	5	16	6	5	8	5	0	8
Rhode Island.....	9	3	16	8	3	13	1	0	3
District of Columbia.....	3	1	2	1	0	2	2	1	0
Nevada.....	2	2	1	0	0	0	2	2	1
Wyoming.....	1	0	1	0	0	0	1	0	1
.....	1	0	1	1	0	1	0	0	0
Total.....	5,808	2,465	6,907	2,897	1,570	3,306	2,907	895	3,597

³ For four accidents the information given was inadequate for even a reasonable guess as to the place of occurrence.

⁴ The registration area comprises those parts of the United States which meet certain standards in the reporting of mortality statistics. During 1925 it included 40 States and 25 cities outside those States (including the District of Columbia), representing 70.9 per cent of the area and 89.4 per cent of the population of the continental United States.

¹ Total includes four accidents unclassified as to rural or urban locality.

total accidental deaths for each city and the deaths resulting from accidents which happened within the city. For the 52 weeks ended November 5, 1927, out of 5,674 deaths reported from 61 of the larger cities, 751 were the result of accidents occurring elsewhere, though not necessarily in rural districts.

Offsetting in some degree this possible 13 per cent error in the statistics for urban fatalities is the fact that in its mortality statistics the Bureau of the Census treats all localities of less than 10,000 population as rural, whereas in its population studies, and in this discussion, the line is drawn at 2,500. Cities of 10,000 and over in the United States include but 42.3 per cent of the entire population, while cities and villages of between 2,500 and 10,000 include 9 per cent. If fatality figures were available separately for the smaller incorporated localities the urban total would be considerably increased.

It therefore appears justifiable to estimate that of all street and highway fatalities, approximately 40 per cent,



A TYPICAL RURAL CROSSING

or 10,000,⁵ in 1926 occurred in rural areas. As shown in Table 1, some 1,570 grade-crossing fatalities occurred in rural regions. On this basis, rural grade crossings are responsible for 16 per cent of all rural highway fatalities.

Two further considerations support the belief that this estimate of 16 per cent is conservative:

(1) As previously stated, the actual number of deaths from grade-crossing accidents includes approximately 8 per cent additional subsequent fatalities.

(2) The grade-crossing accident statistics quoted are only those for the steam railroads of the country. During 1926 there were also 237 persons killed at crossings by the electric railroads which report to the Interstate Commerce Commission. While it is probable that the proportion of urban accidents on these roads is higher than for the steam roads, a considerable number of these fatalities should properly be included in the rural grade-crossing total.

TYPES OF GRADE-CROSSING ACCIDENTS

Of the accidents analyzed in this study, by far the greater number involve passenger automobiles, as might be expected in view of the large predominance of such vehicles. Table 2 shows the number of accidents according to type, together with the number of personal casualties for each type:

TABLE 2.—Grade-crossing accidents by type, and personal casualties, 1926

Type of accident	Total ¹			Rural			Urban		
	Number of accidents	Persons killed	Persons injured	Number of accidents	Persons killed	Persons injured	Number of accidents	Persons killed	Persons injured
Accident involving—									
Passenger automobile	4,035	1,742	5,290	2,122	1,206	2,621	1,910	536	2,666
Automobile truck	880	284	916	468	189	467	411	95	458
Pedestrian	535	301	251	157	104	59	378	197	192
Animal-drawn vehicle	152	48	124	86	34	61	66	14	63
Trolley car	35	1	149	4	0	20	31	1	129
Automobile bus	30	19	89	17	19	53	13	0	36
Motorcycle	17	10	12	6	3	4	11	7	8
Bicycle	10	8	2	1	1	0	9	7	2
Other vehicle	12	3	14	6	2	6	6	1	8
Miscellaneous	102	49	60	30	12	25	72	37	35
Total	5,808	2,465	6,907	2,897	1,570	3,306	2,907	895	3,597

¹ Total includes four accidents unclassified as to rural or urban locality.

The differing significance of rural and urban accidents becomes still more apparent when the accidents are thus classified by type, and the inadequacy of mere numbers of accidents as a measure of grade crossing danger is plain. Deaths for each important type of accident in rural localities are practically double those for urban localities, with the exception of those involving automobile buses, pedestrians and those classed as miscellaneous. In the first-named group all the fatalities occurred in rural accidents, while for the last two types urban accidents predominate. The number of nonfatal injuries is approximately the same for rural and urban accidents in each class except trolley-car and pedestrian accidents. The lower fatality rate in vehicular accidents in urban areas may be attributed, among other things, to the slower speed of both trains and highway vehicles.

The total number of persons killed and injured in practically every group exceeds the number of accidents in that group for the reason that many of the accidents result in plural casualties. Even among the pedestrian accidents there were frequent cases in which two or more persons went upon the track at the same time and were struck by the same train, the several casualties being included in a single accident report.

GRADE CROSSINGS PROTECTED BY VARIOUS DEVICES

A casual study of grade-crossing accidents makes it evident that no sort of crossing protection can guarantee safety to certain sorts of people, nor even, under particular combinations of circumstances, to careful users of the highways. Many grade crossings have been eliminated in recent years from the more important highways either by separation of grade or by relocation of one or both of the rights of way. This remedy is too expensive to be applied everywhere, and it is necessary to resort to various degrees of protection.

The most positive form consists of gates, operated by a watchman on the crossing or in a near-by tower, these gates closing the highway on both sides of the crossing when a train is about to pass. When the gates are down the railroad right of way is fenced off from the public, and highway travelers injured in crossing accidents at such times are classed by the railroads and by the Interstate Commerce Commission as trespassers. The gates may be operated at all times, or only during certain hours when it is believed that the volume of traffic requires them.

⁵ Based on estimate of National Safety Council of 25,000 in 1926.

Next in order is the crossing flagman, who stands on the crossing with a sign or a lantern and stops traffic during train movements. He, like the gateman, may be on duty all or only a part of the 24 hours.

Audible and visual signals are those which, on the approach of a train, sound a bell or other audible signal and further attract attention by a flashing or swinging light or moving sign. They are commonly operated automatically by train movement, like railroad block signals, though, occasionally, manual operation is depended upon.

Crossing bells are also used without a supplementary visual signal (i. e., no lights or moving signal operated by train movement), while visual signals are similarly found without audible accompaniment.

Finally, there are fixed signs, which only call attention to the existence of the crossing, and give no special notice of the approach of trains. Crossings marked by fixed signs only are classed by the Interstate Commerce Commission as "unprotected," and have been so treated in the present study. The fixed sign may be the familiar cross arm or its equivalent, at the side of the road, or it may be some special device such as an overhead sign, a sign placed on an island in the middle of the road, an approach sign placed along the road some distance from the crossing, or a sign painted on the highway surface.



A VILLAGE CROSSING PROTECTED BY AUDIBLE (BELL) SIGNAL AND STANDARD SIGN

MAJORITY OF ACCIDENTS OCCUR AT UNPROTECTED CROSSINGS

In this study the analysis of type of protection is carried farther than in that of the Interstate Commerce Commission, since the kind of protection recorded for each crossing is the highest type normally to be found at that crossing, regardless of its temporary discontinuance for whatever cause. If, for example, an accident occurred at a certain crossing at 11.30 p. m., after the gate operator had gone off duty, that crossing is here tabulated as protected by gates, with the added notation that the gate operator was off duty at the time. Similar treatment is given where gates were reported as out of order, where a flagman was negligent though supposed to be on duty, or where an automatic signal failed to operate. All accidents occurring under such circumstances appear in the reports published by the Interstate Commerce Commission as having taken place at unprotected crossings. On the other hand, it frequently happens that under the rules of some railroads, irregular switching movements are protected by members of the switching crews at crossings otherwise unprotected at the time. In the tabulation here made, and in the reports of the Interstate Commerce Commission, such crossings are reported as protected by

watchmen, even though the regular watchman may be off duty at the time or there may be normally some other type, or no type of protection.

Table 3 shows the frequency of accidents occurring under each type of protection for both rural and urban localities. The great majority of accidents take place at unprotected crossings and it is probable that most of those under the heading "Not stated" are also in this category.



A CROSSING WATCHMAN ON THE JOB

TABLE 3.—Type of protection at grade crossings where accidents occurred in 1926

Type of protection	Total ¹			Rural			Urban		
	Number of accidents	Persons killed	Persons injured	Number of accidents	Persons killed	Persons injured	Number of accidents	Persons killed	Persons injured
Gates.....	412	162	457	60	31	85	352	131	372
Watchman.....	735	236	914	155	69	183	580	167	731
Audible and visual signals.....	220	147	260	115	84	137	105	63	123
Audible signals.....	366	197	414	206	138	203	160	59	211
Visual signals.....	277	123	287	139	82	136	138	41	151
Unprotected.....	2,440	1,101	2,919	1,447	810	1,652	991	291	1,265
Not stated.....	1,358	499	1,656	775	356	910	581	143	744
Total.....	5,808	2,465	6,907	2,897	1,570	3,306	2,907	895	3,597

¹ Total includes four accidents not classified as to rural or urban locality.

Most of the higher types of protection, namely, the gates and watchmen, are in urban localities, while unprotected crossings greatly predominate in the rural accidents. According to figures supplied by the Interstate Commerce Commission, the 235,158 railroad grade crossings on Class I steam railroads⁶ in the United States are protected as shown in Table 4.

NUMBER OF ACCIDENTS NOT A MEASURE OF EFFICIENCY OF PROTECTION

A comparison of Tables 3 and 4 reveals that while less than 3 per cent of the crossings are protected by gates for all or part of the day, 7 per cent of the accidents and almost the same proportion of the fatalities occurred at such crossings. Slightly more than 3 per cent of the crossings are protected by watchmen, but nearly 13 per cent of the accidents and nearly 10 per cent of the fatalities are found at these crossings. This disproportionate number of accidents and fatalities at crossings having the higher types of protection is not to be taken as an indication of the ineffectiveness

⁶ These totals would be little changed by the inclusion of grade crossings on railroads other than Class I carriers.

TABLE 4.—*Kind of protection at highway grade crossings on Class I steam railroads in the United States, 1926*

Kind of protection	Number at beginning of year	Number added during year	Number eliminated during year ¹	Number at end of year
Gates, with or without other protection, operated 24 hours per day.....	3,400	134	187	3,347
Gates, with or without other protection, operated less than 24 hours per day.....	2,986	69	232	2,823
Watchmen, alone or with protection other than gates, on duty 24 hours per day.....	1,263	130	90	1,303
Watchmen, alone or with protection other than gates, on duty less than 24 hours per day.....	6,672	198	408	6,462
Both audible and visual signals, without other protection.....	5,619	959	119	6,459
Audible signals only.....	5,732	80	485	5,327
Visual signals only.....	1,613	685	92	2,206
Special fixed signs or barriers, with or without standard fixed signs.....	26,247	3,634	515	29,366
Standard fixed signs only.....	176,101	3,358	6,205	173,254
Otherwise unprotected.....	4,068	949	406	4,611
Protected.....	27,285	2,255	1,613	27,927
Unprotected.....	206,416	7,941	7,126	207,231
Total.....	233,701	10,196	8,739	235,158

¹ Total includes 195 crossings eliminated by separation of grades.

of such protection. The hazard at any given crossing may be said to be measured roughly by the product of the total highway traffic by the total train movement, although numerous other variables such as topography, highway surface, and speed also play a part. The value of any safeguard must be judged by the number of accidents which are prevented rather than by the number which happen in spite of it. Data of a sufficiently general nature for use in drawing statistical conclusions as to the relative merits of types of crossing protection are as yet not available.

Of the 412 accidents and 162 fatalities which occurred at crossings protected by gates, 58 accidents, with 22 fatalities, occurred during hours when the gate operator was off duty. One hundred and six accidents, with 31 fatalities, occurred when the operator was on duty but failed for one reason or another to have the gates lowered at the proper time, while 220 accidents, with 98 fatalities, occurred in spite of properly lowered gates. These figures are shown in detail in Table 5.

TABLE 5.—*Operation of crossing gates at crossings where accidents occurred, 1926*

Operation of gates	Number of accidents	Persons killed	Persons injured
Gates functioning properly.....	220	98	169
Gateman off duty at hour of accident.....	58	22	86
Gateman on duty but failed to have gates lowered.....	95	29	144
Gates out of order.....	11	2	17
Insufficient information and miscellaneous.....	28	11	41
Total.....	412	162	457

Accidents occurring despite lowered crossing gates were due, with rare exceptions, to vehicles crashing into or through the gates, or pedestrians passing around or under them. Table 6 shows this group in detail.

It will be noted that over half the accidents and more than three-fourths of the fatalities involved pedestrians. Four accidents, with two fatalities, were the result of pedestrians trying to climb between or over cars. In view of the common assertions as to the frequency with which crossing gates are wrecked by motorists, it is of interest to discover that, according to these records, few reportable accidents result. Either the usual

TABLE 6.—*Types of accidents occurring at crossings protected by lowered gates, 1926*

Type of accident	Number of accidents	Persons killed	Persons injured
Collision with passenger automobile.....	67	15	92
Collision with automobile truck.....	7	3	6
Pedestrian accident.....	137	76	65
Miscellaneous.....	9	4	6
Total.....	220	98	169

statements are exaggerated or the resulting casualties are of too minor a nature to come under the reporting rules. Only 74 accidents and 18 fatalities are attributed to motor vehicles at crossings protected by lowered gates.

At crossings protected by watchmen there were 735 accidents and 236 fatalities, of which 130 accidents with 50 deaths occurred during hours when the watchman was off duty. Fifty-four accidents with 32 fatalities were attributable to negligence on the part of the watchman, including 5 cases in which the watchman himself was the victim of a train while on duty. In 504 accidents, taking a toll of 140 lives, the watchman was reported as properly at his post of duty on the highway. Apparently there is sometimes confusion as to the signals given by watchmen to highway travelers, as in a number of instances cars were driven on to the crossing under a mistaken impression that a proceed signal was given. This is especially apt to happen when the watchman is a member of a train crew and is giving signals to his engineman. On the other hand, the watchman is more often completely ignored. Instances were reported in which the watchman was struck by an automobile, or had the lantern knocked out of his hand. One watchman threw his lantern at an approaching car in a last futile effort to bring it to a safe stop.

INTERMITTENT CROSSING PROTECTION GIVES FALSE SENSE OF SECURITY

It should be stated that the absence of the gate operator or watchman does not in every case leave the crossing unprotected. Bells or visual signals are frequently in operation during such hours. Where a gateman or watchman is on duty only a portion of the day, it is customary to post a sign at the crossing indicating the hours during which he is not present. These signs are not always sufficiently conspicuous or legible, and even at best, the person who is familiar with a crossing does not stop to read them or to consult his watch each time he passes. Numerous accidents were noted which occurred within a few minutes after the gateman or watchman went off duty, or just before he arrived on duty. One instance was reported from Arizona in which the motor-vehicle driver was a Chinaman and unable to read the sign which stated that the crossing was unprotected at that hour.

Despite various safeguards, it appears that intermittent protection introduces certain dangers. In the report of the committee on grade-crossing design, protection, and elimination, of the American Railway Engineering Association, December, 1926, this statement is made:

Where gates are in service for less than 24 hours and not in service when there is traffic on the railway, the protection is not satisfactory in that it conveys a false sense of security to those using the crossing and may result in serious injury.

The automatic signals of the audible or visual type appear to be highly dependable in their operation, and there seems to be no reason why they should not be as reliable as the block signals which control train operation. Only 12 cases were found in which they were reported as being out of order. A most serious criticism of such signals, however, is that they will continue to operate as long as there is a train inside the circuit. Where switching is being done in the vicinity it is not uncommon for the bell or other signal to operate continuously for long periods, despite the fact that no train moves over the crossing. At such crossings the stranger is perplexed and the local citizen soon begins to ignore the warnings.

SPEED OF TRAINS

The higher fatality rate in rural accidents appears to be correlated with the higher speed of trains in rural areas. The higher average speed and relatively greater damage done in rural accidents is clearly shown in Table 7.

TABLE 7.—*Speed of train, number of accidents, and casualties at grade crossings, 1926*

Speed of train (miles per hour)	Total ¹			Rural			Urban		
	Number of accidents	Persons killed	Persons in- jured	Number of accidents	Persons killed	Persons in- jured	Number of accidents	Persons killed	Persons in- jured
0 (standing).....	195	18	334	99	12	159	95	6	174
1 to 4.....	509	59	645	128	9	178	381	50	467
5 to 9.....	981	131	1,425	245	36	380	736	95	1,045
10 to 19.....	1,455	378	1,858	613	166	789	841	212	1,068
20 to 29.....	1,014	462	1,206	606	284	733	407	178	472
30 to 39.....	712	550	647	521	391	488	190	159	158
40 to 49.....	479	454	387	360	336	309	119	118	78
50 to 59.....	266	286	194	223	241	167	43	45	27
60 and over.....	84	101	65	65	87	55	19	14	10
Not reported.....	113	26	146	37	8	48	76	18	98
Total.....	5,808	2,465	6,907	2,897	1,570	3,306	2,907	895	3,597

¹ Total includes four accidents not classified as to rural or urban locality.

The 195 accidents occurring when trains were standing still are accounted for principally by collisions of motor vehicles with stationary trains or cars. There were six cases of persons climbing between or over standing cars, resulting in no fatalities but in injuries usually caused by unexpected motion of the cars when the train was coupled on to or started. Perhaps these should not be classed among standing-train accidents. One automobile bus and three trolley cars collided with standing trains, all in urban territory, with no fatalities but with 25 personal injuries. Collisions involving passenger automobiles alone and the casualties for each speed are shown in Table 8.

Chance of survival shows a steady decrease as the speed of the train increases above 10 miles an hour. The slightly higher mortality rate for the very slowest train movement is probably explained by the high proportion of automobiles running into trains in the low-speed groups. It is usually the reckless and high-speed driver who suffers from this type of accident, and what the train lacks in momentum, he supplies.

Pedestrian accidents usually occur near stations or in yards where switching is being done on multiple tracks. This is reflected in the speed of trains involved in such accidents. Excluding those accidents due to persons climbing between or over cars, the pedestrian accidents and speed of trains are shown in Table 9.

TABLE 8.—*Speed of train, accidents, and casualties for collisions involving passenger automobiles, 1926*

Speed of train (miles per hour)	Number of accidents	Persons killed		Persons injured	
		Total	Per acci- dent	Total	Per acci- dent
0 (standing).....	172	17	0.10	284	1.65
1 to 4.....	334	29	.09	459	1.37
5 to 9.....	666	51	.08	1,070	1.61
10 to 19.....	985	234	.24	1,418	1.44
20 to 29.....	715	341	.48	942	1.32
30 to 39.....	510	422	.83	516	1.01
40 to 49.....	335	347	1.04	294	.88
50 to 59.....	188	219	1.16	146	.78
60 and over.....	58	70	1.21	54	.93
Not reported.....	72	12	.17	107	1.49
All speeds.....	4,035	1,742	.43	5,287	1.31

TABLE 9.—*Showing speed of train, accidents, and casualties involving pedestrians, 1926*

Speed of train (miles per hour)	Number of accidents	Persons killed	Persons in- jured	Speed of train (miles per hour)	Number of accidents	Persons killed	Persons in- jured
0 (standing).....	1	1	0	40 to 49.....	41	39	4
1 to 4.....	56	16	42	50 to 59.....	19	17	5
5 to 9.....	117	49	73	60 and over.....	5	4	1
10 to 19.....	150	80	72	Not reported.....	13	7	6
20 to 29.....	69	46	24				
30 to 39.....	41	38	5	All speeds.....	512	296	233

¹ 1 pedestrian injured himself by stumbling against a standing train.



A CROSSING WITH AN UNUSUALLY RESTRICTED VIEW, PROTECTED BY AUDIBLE AND VISUAL SIGNALS—BELL AND SWINGING DISK

GRADE-CROSSING ACCIDENTS VARY WITH SEASON OF YEAR

There is a marked seasonal variation in grade-crossing accidents as shown in Table 10 and Figure 1. Surveys made by the Bureau of Public Roads on the highways of several eastern States have shown August to be the month of maximum traffic. Nevertheless, rural grade-crossing accidents do not reach their peak until October, and urban accidents until November, with a marked peak in November for both combined. For fatalities the maximum is reached in October. The only reasonable explanation of this condition appears to be that people are slow in adjusting their walking and driving habits to meet the inclement winter weather. Slippery roads, side curtains or frosty windows, rainy windshields, early darkness, umbrellas—these and other factors doubtless play their part.

A relative increase in recent years in the number of grade-crossing casualties occurring during the late fall months is perhaps due to improved roads, the more general use of closed cars and the consequent increasing volume of winter traffic. The figures published by the

TABLE 10.—*Grade crossing accidents and personal casualties by months, 1926*

Month	Total ¹			Rural			Urban		
	Number of accidents	Persons killed	Persons injured	Number of accidents	Persons killed	Persons injured	Number of accidents	Persons killed	Persons injured
January.....	444	178	511	208	98	248	236	80	263
February.....	436	163	519	195	105	216	241	58	303
March.....	435	170	517	219	118	230	216	52	287
April.....	409	152	473	208	101	208	200	51	264
May.....	432	186	532	205	113	240	227	73	292
June.....	436	204	502	229	131	264	206	73	237
July.....	414	222	527	225	135	306	189	87	221
August.....	441	187	516	236	128	247	204	59	268
September.....	463	214	557	257	146	314	206	68	243
October.....	624	287	736	318	185	363	305	102	372
November.....	666	255	813	312	154	370	354	101	443
December.....	608	247	704	285	156	300	323	91	404
Total.....	5,808	2,465	6,907	2,897	1,570	3,306	2,907	895	3,597

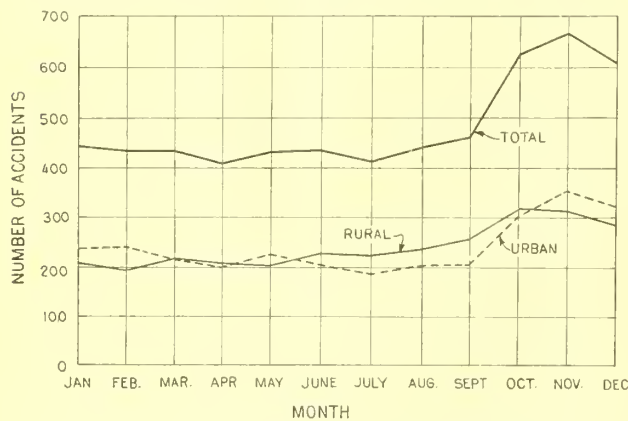
¹ Total includes four accidents not classified as to rural or urban locality.

FIG. 1.—GRADE-CROSSING ACCIDENTS BY MONTHS IN 1926

Interstate Commerce Commission show August as the month of maximum fatalities in 1921, 1922, and 1923, November in 1924, and October in 1925 and 1926. For fatal and nonfatal injuries combined, the peak was reached during October in 1921 and 1922, during November in 1923, 1924, and 1926, and during December in 1925.⁷

GRADE-CROSSING ACCIDENTS BY DAYS OF THE WEEK

Most persons, if told that fewer grade-crossing accidents occur on Sunday than on any other day, would express considerable surprise. Such is the fact, however, as revealed by this study. In number of accidents Saturday is far ahead of any other day.⁸ The statistics for grade-crossing fatalities, however, are considered more significant than those for number of accidents, and these tell a different story. In fatalities Sunday heads the list, with Saturday only a close second. The mortality per accident, therefore, reaches its peak on Sunday. This is the day when the whole family and some of the neighbors crowd into the car for a trip out into the country. The newspapers on Monday morning frequently report the deaths of entire families at grade crossings.

⁷ See Accident Bulletin No. 94, Bureau of Statistics, Interstate Commerce Commission, p. 18.⁸ In New York State during 1926, according to the report of the State bureau of motor vehicles, the maximum number of all types of motor vehicle accidents and fatalities occurred on Saturday, with Sunday in second place.

The number of grade-crossing accidents and personal casualties by days of the week are shown in Table 11 and Figure 2.

TABLE 11.—*Grade-crossing accidents and personal casualties by days of the week, 1926*

Day	Total ¹			Rural			Urban		
	Number of accidents	Persons killed	Persons injured	Number of accidents	Persons killed	Persons injured	Number of accidents	Persons killed	Persons injured
Sunday.....	713	454	1,029	379	314	506	334	140	523
Monday.....	825	325	919	409	205	452	415	120	466
Tuesday.....	842	309	948	408	181	437	433	128	510
Wednesday.....	833	295	964	415	194	471	418	101	493
Thursday.....	776	279	850	382	172	392	393	107	457
Friday.....	842	355	942	416	215	479	426	140	516
Saturday.....	976	448	1,200	487	289	568	488	159	632
Total.....	5,807	2,465	6,852	2,896	1,570	3,305	2,907	895	3,597

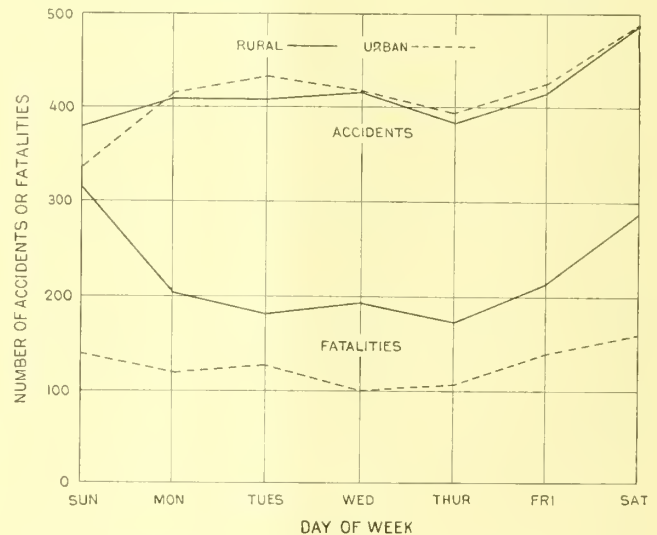
¹ Total includes four accidents unclassified as to rural or urban locality.² Does not include one rural accident for which exact date was not reported.

FIG. 2.—GRADE-CROSSING ACCIDENTS AND FATALITIES BY DAYS OF THE WEEK, 1926

The heavy toll on pleasure driving is shown even more emphatically in the figures for rural grade-crossing accidents involving passenger vehicles in the summer months, in Table 12 and Figure 3. During September and October there were twice as many fatalities on Sunday as on any other day.

TABLE 12.—*Rural grade-crossing accidents and fatalities for passenger automobiles by days of the week during the summer months, 1926*

Day	6 months, May to October, inclusive		2 months, September and October	
	Number of accidents	Persons killed	Number of accidents	Persons killed
Sunday.....	181	190	80	97
Monday.....	137	70	44	15
Tuesday.....	138	62	50	24
Wednesday.....	144	71	50	28
Thursday.....	137	69	57	31
Friday.....	146	73	60	26
Saturday.....	195	119	76	44
Total.....	1,078	654	417	265

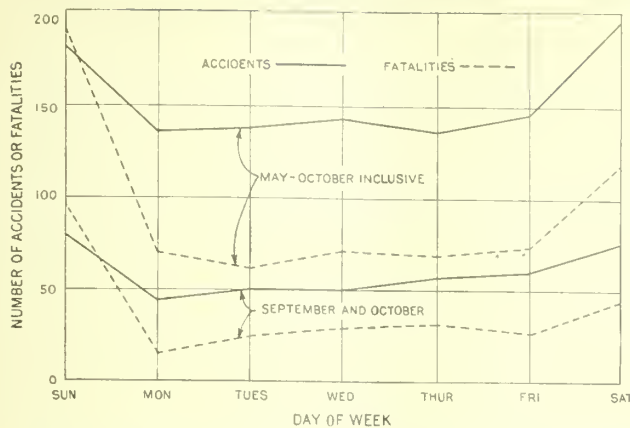


FIG. 3.—RURAL GRADE-CROSSING ACCIDENTS AND FATALITIES FOR PASSENGER AUTOMOBILES BY DAYS OF THE WEEK DURING THE SUMMER MONTHS, 1926

The severity of these Sunday accidents is apparent from the fact that the number of deaths exceeds the number of accidents. While these average more than one fatality per accident, passenger-automobile grade-crossing accidents throughout the year average less than 1 fatality for every two accidents.⁹

GRADE-CROSSING ACCIDENTS BY HOURS

Published accident statistics show that the dangerous hour in traffic comes in the late afternoon when a presumably fatigued populace is hastening home from work. Such figures, however, are predominantly from city reports or from the more urban States, rural accident reporting being as yet in a feeble but not unpromising infancy. It may, therefore, be significant that while grade-crossing accidents in general, and urban grade-crossing accidents in particular, reach a peak during the hour between 5 and 6 p. m., rural grade-crossing accidents reach their peak between 3 and 4 p. m. The figures are given in Table 13, and shown graphically in Figure 4.

TABLE 13.—Grade-crossing accidents and personal casualties by hours, 1926

Hour	Total ¹			Rural			Urban		
	Number of accidents	Persons killed	Persons injured	Number of accidents	Persons killed	Persons injured	Number of accidents	Persons killed	Persons injured
12 to 1 a. m.	155	47	231	55	19	64	100	28	167
1 to 2 a. m.	112	25	174	44	16	65	68	9	109
2 to 3 a. m.	73	16	103	26	5	38	47	11	65
3 to 4 a. m.	61	12	84	21	4	27	40	8	57
4 to 5 a. m.	58	24	65	21	6	24	37	18	41
5 to 6 a. m.	100	29	118	36	15	38	64	14	80
6 to 7 a. m.	211	70	254	81	37	92	130	33	162
7 to 8 a. m.	260	92	284	137	62	160	123	30	124
8 to 9 a. m.	271	119	287	152	81	167	119	38	120
9 to 10 a. m.	288	131	286	166	84	171	121	47	114
10 to 11 a. m.	329	139	335	192	102	181	137	37	154
11 to 12 a. m.	332	161	368	202	129	217	130	32	151
12 to 1 p. m.	318	152	367	177	106	192	141	46	175
1 to 2 p. m.	290	136	315	156	89	165	133	47	149
2 to 3 p. m.	326	179	341	195	138	202	131	41	139
3 to 4 p. m.	373	199	395	241	150	258	132	49	137
4 to 5 p. m.	349	187	385	192	134	202	157	53	183
5 to 6 p. m.	410	205	463	183	87	224	227	118	239
6 to 7 p. m.	341	141	396	161	90	191	180	51	205
7 to 8 p. m.	282	115	350	123	64	148	159	51	202
8 to 9 p. m.	231	70	354	89	35	121	142	35	233
9 to 10 p. m.	210	62	311	90	42	134	119	20	176
10 to 11 p. m.	223	74	316	72	42	94	150	32	221
11 to 12 p. m.	193	79	306	77	32	117	116	47	189
Hour not stated...	12	1	19	8	1	14	4	0	5
Total.....	5,808	2,465	6,907	2,897	1,570	3,306	2,907	895	3,597

¹ Total includes four accidents not classified as to rural or urban locality.

⁹ See Table 2.

The curves for hourly fatalities (not shown) closely correspond with those for numbers of accidents, though, of course, on a lower level.

Pedestrian accidents in urban localities show a very marked peak between 5 and 6 p. m., as will be seen in Figure 5 and Table 14. For passenger automobiles only in urban crossing accidents the peak is reached between 10 and 11 p. m., although the fatalities show a decided maximum between 5 and 6 p. m. (Fig. 6 and Table 14.)

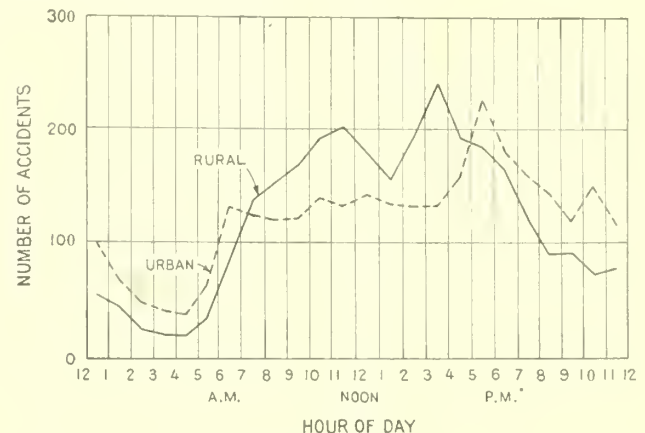


FIG. 4.—RURAL AND URBAN GRADE-CROSSING ACCIDENTS BY HOURS, 1926

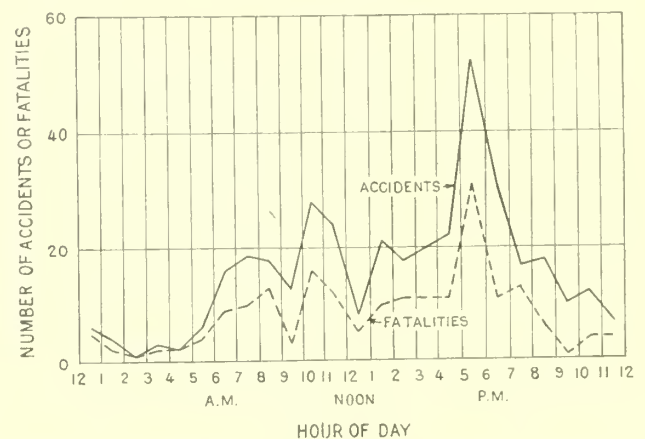


FIG. 5.—PEDESTRIAN ACCIDENTS AT URBAN CROSSINGS, BY HOURS, 1926

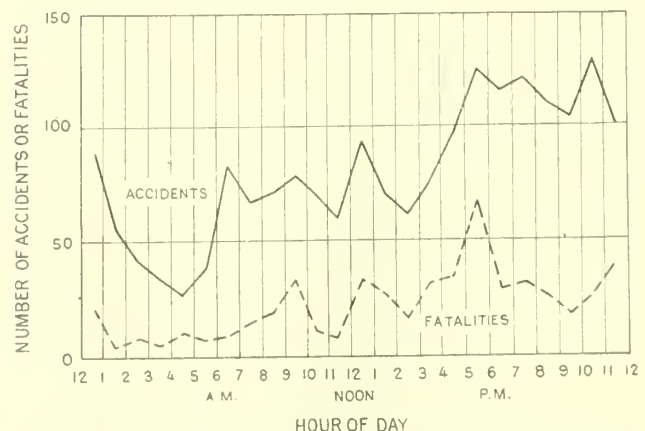


FIG. 6.—PASSENGER-AUTOMOBILE ACCIDENTS AT URBAN GRADE CROSSINGS, BY HOURS, 1926

TABLE 14.—*Pedestrian and passenger automobile accidents and fatalities at urban crossings, by hours, 1926*

Hours	Pedestrian accidents		Passenger auto accidents	
	Number of accidents	Persons killed	Number of accidents	Persons killed
12 to 1 a. m.	6	5	88	21
1 to 2 a. m.	4	2	54	5
2 to 3 a. m.	1	1	41	9
3 to 4 a. m.	3	2	33	6
4 to 5 a. m.	2	2	27	11
5 to 6 a. m.	6	4	38	8
6 to 7 a. m.	16	9	82	10
7 to 8 a. m.	19	10	66	16
8 to 9 a. m.	18	13	71	20
9 to 10 a. m.	13	3	78	33
10 to 11 a. m.	28	16	69	12
11 to 12 a. m.	24	12	59	8
12 to 1 p. m.	8	5	94	33
1 to 2 p. m.	21	10	70	27
2 to 3 p. m.	18	11	61	17
3 to 4 p. m.	20	11	75	31
4 to 5 p. m.	22	11	96	34
5 to 6 p. m.	52	31	126	66
6 to 7 p. m.	31	11	116	28
7 to 8 p. m.	17	13	122	32
8 to 9 p. m.	18	6	110	26
9 to 10 p. m.	10	1	103	18
10 to 11 p. m.	12	4	130	26
11 to 12 p. m.	7	4	100	39
Hour not stated	2	0	1	0
Total	378	197	1,910	536

VISIBILITY OF APPROACHING TRAINS DISCUSSED

A study was also made of the degree to which the approaching train was visible to the traveler on the highway. Accidents do happen under the most favorable conditions of visibility—almost incredible accidents for which the only reasonable explanation is sheer inattention. Nevertheless, it is equally certain that many of the mishaps reported would not have occurred had the vehicle operator been able to see the approaching train a moment sooner than he did. The average motorist slows down when he sees the railroad crossing sign ahead, but he is disinclined to stop until he sees the necessity for so doing. Stopping and starting a car causes a few moments of delay, and calls for a certain physical and nervous effort, especially when the highway leads upgrade to the crossing. Where the driver does not have a clear view up and down the track for some distance before he reaches the rails, he is apt to see the train too late for a safe stop in the distance left to him.

A recent decision of the United States Supreme Court¹⁰ places the responsibility for caution at grade crossings squarely upon the motorist or other user of the highway. Under this decision the motorist is expected to take every care to learn whether the way is clear, even, when necessary, to the extent of getting out of his car for a look up and down the track. Even with the responsibility so definitely assigned, however, the improvement of visibility at grade crossings should be as much a part of a highway-safety program as any other measures designed to protect the careless driver and to expedite traffic generally.

The ideal grade crossing, as measured by the criterion of visibility of approaching trains, would be in the center of a wide open field. The traveler nearing it from either direction on the highway would have a clear view, beginning several hundred feet away and continuing until he reached the crossing, of any train within a distance of at least a thousand feet. Even a

fast driver arriving as the limited express was due would have ample time and space in which to stop safely under such conditions. And yet accidents happened in 1926 at crossings answering practically to this description. As a rule, however, there are buildings or other obstructions to clear visibility located, not far from the track. The illustration below shows an urban crossing where the view is definitely cut off up to a few yards from the rails. Approximately 35 trains pass this point daily, some of them fast expresses. Two women in an automobile were killed here during 1927.

There is no standard or generally accepted definition of what constitutes a reasonable and safe view at a



AN UNPROTECTED URBAN CROSSING. THE FIXED RED LANTERNS GIVE NO SPECIAL WARNING OF APPROACHING TRAINS. THIS WAS THE SCENE OF A RECENT FATAL ACCIDENT

crossing. This probably explains why the railroad reports were rarely specific in respect to the conditions of visibility. Where the view was mentioned at all (and there were only 3,224 such reports altogether) the information was usually limited to the statement that the view was "clear" or "good," or that it was obstructed in one way or another. Some of the reporting officers appeared to be using the term "unobstructed view" to mean that the traveler was able to see the crossing ahead, without regard to the visibility of any approaching train.

Sometimes the view was described as "clear," with a following statement indicating that obstructions were located perhaps as near as 50 feet from the track. The difficulty of bringing such reports to a common basis is apparent. Some of the roads, however, carefully reported just how far from the crossing, measured along the highway, the train would be visible for a named distance as it approached on the rails. One of the smaller roads sent in with each of its reports a detailed blue print showing the exact relation of highway to railroad, including the grade of each, as well as the location of all buildings, trees, and other obstructions to a clear view. The exact distances at which the train might be seen from different points on the highway were also indicated.

Table 15 shows the result of the analysis of the conditions of visibility at crossings where accidents occurred involving motor vehicles. In relation to accidents involving slower vehicles or pedestrians, obstructions to view are of less importance or play no part. The category, "clear," in this table includes all those in-

¹⁰ Baltimore and Ohio Railroad Co. v. Dora Goodman, October 31, 1927.

stances in which the view was reported as "clear" or "good," without further qualifications. It also includes those in which visibility was reported as clear to at least 100 feet from the rails. Those classed as having an obstructed view are all those at which some obstruction was reported within the 100-foot zone, as well as those for which the distance of the given obstruction was not specified. The 100-foot limit was chosen as the minimum which could be regarded as giving a safe view at modern highway speeds. In each case only the most important obstruction was tabulated.

TABLE 15.—*Visibility of approaching trains in motor-vehicle grade-crossing accidents, 1926*

Conditions of visibility	Number of accidents		
	Total ¹	Rural	Urban
Clear.....	1,818	1,054	763
Obstructed by building.....	395	182	213
Obstructed by vegetation or bank.....	160	128	31
Obstructed by train or railroad car.....	195	82	113
Obstruction not stated.....	307	154	153
Conditions not stated.....	2,087	1,013	1,072
Total.....	4,962	2,613	2,345

¹ Total includes four accidents not classified as to rural or urban locality.

For but 117 rural accidents and 65 urban accidents was it definitely stated that there was a clear view of the approaching train at all points within 100 feet of the crossing in the direction from which the vehicle approached. There were doubtless numerous other crossings where this condition also existed but was not specifically reported. Often, too, the obstruction to view caused by a building or railroad car, for example, was only partial, the vehicle operator having his view cut off for only a short while after having had a clear view at a safe distance previously.

Most of the instances of obstruction to view caused by trains or railroad cars are chargeable to standing freight cars on parallel tracks, though in occasional cases the victims were reported to have passed behind one moving train into the path of another. Often, especially at urban crossings, the cars are only one factor of several which combine to cut off the view, but the most serious because nearest to the track of the oncoming train. Almost every motorist can recall with uneasiness driving over some utterly blind crossing where a string of box cars was standing on a track immediately adjacent to that on which another train might be approaching. Even when the cars are some distance from the highway crossing they may constitute a serious obstruction to any adequate view.

TRAIN VERSUS AUTOMOBILE

In its published accident bulletins, the Interstate Commerce Commission presents an analysis showing separately those cases in which persons or vehicles were struck by trains at crossings and those in which they ran into the side of trains which were already on the crossing. These figures are frequently quoted, usually as an impressive illustration of the carelessness of highway users. While it is true that a train already occupying a crossing is not easily overlooked, is equally true that some trains arrive on the scene rather suddenly. At a mile a minute, a train travels 88 feet in a second. At even moderate speeds, the difference of a very small fraction of a second may deter-

mine whether an automobile is struck by the engine or whether it strikes the side of the engine at the pilot beam, the cylinder, or the drivers. The degree of responsibility for a collision can hardly be significantly affected by a distinction measured in such brief intervals of time. On the other hand, when a motorist strikes somewhere far back on a long train, or collides with a standing or slowly moving car or train, there is reason to suspect excessive speed, inattention, or other fault on the part of the driver.

Of the 4,962 accidents involving motor vehicles here tabulated, 1,110 were reported as cases in which the



THE SAFE SUCCESSOR TO A DANGEROUS GRADE CROSSING ON AN IMPORTANT HIGHWAY

vehicle ran into the side of the train. Further analysis reveals the fact that at least 513 of these collisions occurred when the motor vehicle struck the engine, or the leading car of a train not preceded by an engine. In only 289 cases was it definitely reported that the train was struck somewhere behind the engine or leading car. While this crude analysis takes no account of the speed of the train, it will be apparent that too sweeping conclusions should not be drawn from the fact that highway vehicles—and pedestrians—¹¹ frequently collide with the sides of trains.

DATA AS TO VEHICLE OPERATORS INADEQUATE

While some of the reporting railroads were careful to give the age and other personal facts concerning vehicle operators and pedestrians involved in crossing accidents, the data are far from complete. Even with the best of intentions it is not always possible after a serious automobile accident to learn which of several occupants was driving at the time. In the analysis here made, victims were presumed to be adults unless there was definite evidence to the contrary or ground for reasonable doubt. Children are those reported as such or those whose ages were given as under 18 years.

Without comparable data as to the ratios of men, women, and children operating motor vehicles on the streets and highways, this table can not be used as the basis for any conclusions as to the relative safety of the different classes of operators. It may be suggested, however, that the 50 motor-vehicle accidents occurring to child operators out of a total of 4,962 seems disproportionately high. The larger proportion of child operators in the rural accidents, too, is noticeable, and

¹¹ In 25 cases during 1926.

TABLE 16.—Operators of motor vehicles and pedestrians involved in grade-crossing accidents, 1926

Class	Total ¹		Rural		Urban	
	Motor-vehicle operators	Pedestrians	Motor-vehicle operators	Pedestrians	Motor-vehicle operators	Pedestrians
Adult—male.....	2,914	330	1,543	90	1,368	240
Adult—female.....	251	128	134	38	117	90
Child.....	50	59	35	22	15	37
Not stated.....	1,747	18	901	7	845	11
Total.....	4,962	535	2,613	157	2,345	378

¹ Total includes four accidents not classified as to rural or urban locality.

is probably to be attributed to the absence of regulations, or the more liberal regulations regarding the licensing of operators in the less urbanized States, together with the laxity of enforcement of such regulations in the rural areas even where they do exist.

One hundred and three intoxicated motor-vehicle operators were reported and 25 intoxicated pedestrians. It is not surprising to discover that of the 103 drivers, 64 were listed as having run into the train and only 39 as having been struck by the train. The high proportion of the former may be due to the desire of the reporting officers to explain certain otherwise inexplicable accidents, whereas in more ordinary cases intoxication might be passed over without special mention.

Twenty-four motor-vehicle operators, seven drivers of animals, and forty-seven pedestrians were reported as partially or totally deaf. Of the pedestrians a very considerable proportion were quite aged and sometimes of failing mentality.

INFLUENCE OF WEATHER AND LIGHT DISCUSSED

The importance of climatic conditions and light as factors in the grade-crossing accident rate can not be determined without a knowledge of the relative prevalence of the different sorts of weather and the volume of highway traffic under each. That unfavorable weather does bring more than its share of accidents, however, seems to be indicated by the figures shown in Table 17.

TABLE 17.—Weather and light conditions at time of grade-crossing accidents, 1926

Weather and light	Number of accidents		
	Total ¹	Rural	Urban
Clear, daylight.....	3,056	1,696	1,360
Cloudy, daylight.....	498	281	215
Foggy, daylight.....	44	32	12
Raining, daylight.....	173	102	71
Snowing, daylight.....	69	36	33
Clear, dark.....	1,392	520	870
Cloudy, dark.....	283	114	169
Foggy, dark.....	70	36	34
Raining, dark.....	158	53	105
Snowing, dark.....	65	27	38
Total.....	15,808	2,897	2,907

¹ Total includes four accidents not classified as to rural or urban locality.

OTHER CIRCUMSTANCES

Various other details were noted and recorded in the course of this inquiry, some of which in the final analysis proved to be either insignificant, incomplete, or of doubtful accuracy. A study of the angle of intersection and the highway grade at the crossings, for example, was abandoned for lack of adequate information in a very large majority of the reports. In view of the nature of grade-crossing accidents, too, accurate statements as to the equipment and condition of vehicles and the mental processes of their operators are sometimes unobtainable.

Defective brakes were mentioned as contributing factors in only 51 motor-vehicle accidents. Inadequate automobile headlights were reported in only 14 instances.

Attempts to beat the train were occasionally reported, but usually only as the opinion of the reporting officer. The reasons for many of the nearly 250 cases in which the drivers apparently stalled on the crossings can never be known. In no case was a motor-vehicle operator said to have been asleep at his wheel.

A question of definition is raised by the fact that electric or gasoline rail cars or trains (not including section motor cars) operated by the steam railroads figured in 157 of the accidents. From one viewpoint it can be argued that these should be segregated from the other accidents on steam roads.

HIGHWAY RESEARCH BOARD HOLDS ITS SEVENTH ANNUAL MEETING

Reported by A. C. ROSE, Associate Highway Engineer, United States Bureau of Public Roads

REPORTS on the year's progress in highway research were presented to a group of investigators prominent in the field of research at the seventh annual meeting of the Highway Research Board of the National Research Council. The meeting was opened by the presiding officer, T. R. Agg, of Iowa State College, on December 1, at the National Academy of Sciences Building, Washington, D. C., and Vernon Kellogg, permanent secretary of the National Research Council, delivered a brief address of welcome to the visiting delegates from the several State highway departments, universities, and colleges, and the industries.

VALUE OF RESEARCH DISCUSSED

Frank B. Jewett, the retiring chairman of the division of engineering and industrial research, discussed the value of scientific investigation to industry from a retrospective and prospective point of view. Referring to the necessity for a rigid control of the variables as a fundamental of all scientific research, the speaker stated that there is a tendency to draw conclusions in regard to one factor without giving proper consideration to the other controlling factors. In order to reduce this error to a minimum, the work should be controlled so that only one factor in the problem is a variable. With two variables the interpretation of the data becomes very difficult and with three or more variables it is practically impossible to reach accurate conclusions.

Research workers, continued Mr. Jewett, are prone to try initial experiments on too large a scale before the feasibility of the project has been established in a small way. When the large-scale projects are carried out without the necessary preliminary test tube and other work in the laboratory, it is often difficult to determine the exact reason why satisfactory results are not obtained. For satisfactory progress in research the following routine procedure was advanced as necessary to the solution of each individual problem:

1. Make a decision as to the essentials of the problem.
2. Try out the experiment on a laboratory scale, with the work carefully controlled so as to eliminate all but a single factor in any given experiment.
3. If experimentally successful in the laboratory, then due consideration should be given to the transfer of the research to a larger scale.
4. The large-scale test usually indicates that some modifications are necessary before the apparatus may be used commercially.
5. The next step is to manufacture the tool-made apparatus so as to determine whether large-scale production may be economically feasible.

It is only after these five routine operations have been carried out that research workers are able to broadcast with assurance the successful conclusion of an experiment. This method has been in use in the telephone industry for the past 20 years.

Another lesson that research workers are learning, said Mr. Jewett, is that an infinitesimally small amount of a given chemical substance may produce stupendous changes in the character of the aggregate material. As an illustration of this fact, he cited the successful search some years ago for an alloy that would exhibit

greater magnetic properties than any substance then in existence. This was found by research to be an alloy now known as "permalloy," consisting of iron and nickel, which showed magnetic characteristics 100 times greater than any other known material. It was later discovered that one-thousandth to one-hundredth of 1 per cent of a certain so-called "poisonous" material would reduce the magnetic properties to 10 per cent of what they were in the pure alloy.

It has also been found, in connection with studies of the ravages of teredos on wooden piles, that the life of concrete in sea water varies considerably. This variation does not seem to be due to the character of the concrete mixture but rather to the chemical composition of the aggregate. Such experiences as these, Mr. Jewett said, have led him to the conclusion that in making combinations of various materials, as is necessary in road-building operations, due consideration should be given to the chemical reactions of the aggregate.

In summing up his comments with regard to research in retrospect, Mr. Jewett emphasized the necessity of basing conclusions:

1. On good engineering practice with the experiments controlled so as to confine the data to one variable.
2. On the constituents of which engineering articles are constructed. Information should be gathered concerning different proportions of the individual components of the aggregate material.

With regard to prospective research, viewing the situation as a whole, Mr. Jewett pointed out that pure and applied science is producing a rapidly accumulating supply of knowledge. The universities are turning out more and more fundamental information and this is one of their two primary jobs. The other vital work of the higher institutions of learning—as important as delving for facts—is that of supplying highly trained individuals. The research branches of commercial organizations are applying the new store of data to the solution of practical problems. For this reason more rapid progress may be expected in the decades to come than has been possible in the past.

REPORT OF THE DIRECTOR

Charles M. Upham, director of the Highway Research Board, presented a report of activities during the past year. After giving a historical review of the National Research Council, beginning with its creation in 1916 by an Executive order of President Wilson, Mr. Upham stated that the Highway Research Board was organized as a branch of the national body in 1920, with officers and six research committees.

Examples where industry has profited by discoveries in pure science were presented and in these discoveries the chemist was an important factor. Analogy was made between these problems and the needed research on subgrade clay.

At present there are eight committees, which the director reported are making excellent progress, but more could be done if more ample funds were available. He also suggests as desirable the appointment of paid secretaries for each committee who could devote their

entire time to the work. This suggestion was advanced, not to minimize the excellent work already accomplished by the chairmen of the existing committees, but as a means of obtaining greater results.

MOTOR-VEHICLE ACCIDENTS STUDIED

Austin B. Fletcher, chairman of the committee on causes and prevention of highway accidents, in his introductory remarks, preliminary to the reading of the detailed papers by the six members of his committee, stated that motor-vehicle accidents were increasing at the rate of 5 per cent a year, and for this reason it did not seem necessary to emphasize the need of highway-safety precautions. In order to make the investigations of the committee as comprehensive as possible, there were included in its membership not only highway engineers but also a psychologist, a physicist, and an automobile manufacturer.

S. J. Williams, director of the public-safety division of the National Safety Council, a member of the committee, estimated motor-vehicle fatalities in 1926 at 23,000, and stated that they are growing at the rate of 1,000 a year. In the cities two-thirds or more of the accidents involve pedestrians, while on the rural roads the corresponding proportion is one-half. The speaker stressed the need for more accurate information concerning the causes of accidents since, unfortunately, only about six States require systematic accident reports and statistics. To supply this deficiency, Mr. Williams advocated the adoption of a standard reporting system that would consist of: (1) An individual report card, and (2) a tabulation form for summarizing the cards monthly and annually. For small municipalities, tally sheets would be used, and for large States the punch-card system would be more economical.

In trying to arrive at the causes of accidents Mr. Williams believes it is a mistake to try primarily to fix the personal responsibility. He thinks it a much better plan to collect data as to the circumstances or facts in each case in order to eliminate the element of personal judgment. He concludes, however, that the most refined accident statistics are at best only the first step in solving the problem, the final solution of which depends upon the application of remedial measures, based upon a scientific study of the accident statistics.

M. G. Lloyd, chief of the safety section of the United States Bureau of Standards, delivered a paper on the standardization of motor-vehicle equipment as a means of reducing the accident risk. He believes that headlight devices should be regulated by the States and not by Federal or municipal authorities.

The general specifications now in use are those adopted by the Illuminating Engineering Society in 1920, and revised in 1922. These have also been adopted by the engineering standards committee. There are two basic ideas included in these specifications:

1. An attempt to avoid throwing excessive light into the eyes of the driver of an approaching automobile.
2. An attempt to throw the maximum amount of light upon the highway.

The defect in the present regulations is that while they require adequate equipment they do not include sufficiently drastic regulations concerning the maintenance of the apparatus in proper adjustment. This deficiency, said Doctor Lloyd, should be remedied. The depressible beams of light, now being developed commercially, involve legal complications because they are in conflict with the existing laws based upon fixed-

beam lights. The adjustment of headlights involves two factors: (1) Proper aiming, and (2) proper focusing.

The speaker continued with a discussion of tail lights and rear signaling lamps and concluded with a reference to the new code which had been prepared for testing and regulating brakes. With the four-wheel brakes in use, Doctor Lloyd believes the requirements relative to stopping distance could be made more severe.

PSYCHOLOGY A FACTOR IN ACCIDENT PREVENTION

In delivering the third section of the accident report, Knight Dunlap, professor of experimental psychology at Johns Hopkins University, said that psychologists were interested in the sense of perception and the formation of habits. It is important in reducing accidents to develop safety habits which become instinctive. The time lost in thinking of the action required is often critical. Unnecessary and misplaced signs cause accidents not only at the point at which they are situated but also at other locations, because they breed a contempt of warning signs which leads to the development of unsafe driving habits.

In commenting upon the proper colors for luminous signals and fixed signs, Doctor Dunlap stated that the entire retina of the eye is not equally susceptible to color. For all practical purposes, the rapid discernment of color is limited to the central vision. Yellow and blue are excellent colors for this purpose, but the generally used red and green are tolerably satisfactory. In order to improve the colors now generally accepted, the green should be made as blue as possible, and then, if the cautionary yellow color could be abandoned, it would be better to shade the present red into an orange-red color.

With regard to refusing licenses to color-blind people, the speaker said it is impracticable to solve the problem by refusing to issue licenses. Color-blind persons with some preliminary training can pass a casual examination and only a thorough investigation by a highly trained physician or psychologist can detect vision defective in regard to color. Furthermore, normal persons, when under the strain of an examination, have their color sense sometimes temporarily disturbed.

John C. Long, secretary of the street-traffic committee of the National Automobile Chamber of Commerce, gave a report of a study limited to motor-truck accidents in Hartford, Conn., during the period of January 1 to 30, 1927. All accidents involving property damage of \$10 or more were included. The conclusions show that drivers were at fault in 57 per cent of the cases. Difficulty was experienced in obtaining information because of the fear of unfavorable publicity on the part of the motor-truck owners. This difficulty was removed when it was explained that the aim was not to fix personal responsibility but to determine the causes of accidents and to reduce these by adequate instruction in a motor-truck drivers' school.

A. N. Johnson, dean of the University of Maryland, showed lantern slides giving the results of a method used for measuring the relative efficiency of traffic flow through street intersections under various conditions of control, such as traffic lights and police officers. The speaker believed that the flow of traffic through street intersections is a fundamental consideration both with respect to safety and to the speeding up of traffic.

W. G. Eliot, 3d, of the Bureau of Public Roads, concluded the accident reports with a statistical study of the number of grade-crossing accidents in the United States. The data indicated that grade cross-

ings are responsible for 10 per cent of all motor-vehicle fatalities, but only 1 per cent of the total injuries. The number of rural and urban grade-crossing accidents are almost equal, but 65 per cent of the fatalities are rural. One explanation of this is the slower speed of trains in cities. Mr. Eliot estimates that only 40 per cent of traffic accidents of all kinds occur on rural highways, and that grade-crossing accidents represent 16 per cent of the total on rural roads.

PROGRESS MADE IN VARIOUS STUDIES RELATING TO VEHICLE OPERATION

Professor Agg, the chairman of the committee on economic theory of highway improvement, in summarizing the reports of individual members of the committee, stated that the paper on the measurement of tractive resistance, although not perhaps a final report, may be considered to represent the completion of the project. Roughly, the report dealt with four factors: (1) Rolling resistance, influenced by the condition and kind of tire; (2) impact, as affected by the distortion or character of the road surface; (3) air resistance; and (4) the coefficient of friction between the tire and the roadway surface.

Relative to the progress report on the cost of operation of automobiles, Professor Agg stated that the records show lower costs per mile as compared with figures obtained four or five years ago. This was believed to be caused by the greater distance traveled annually by the average driver and also to the improved character of motor vehicles.

It was announced that the report on the effect of roadway surface on tire wear was not ready for presentation but that it was expected to be completed in time to be made a part of the printed proceedings.

Professor Agg referred to studies his committee has made on the contribution of common-carrier busses to State-road funds. The information so far tabulated had been obtained from the records of the public service commissions of Michigan and Iowa. In Iowa the contribution of the motor bus per ton-mile is decidedly greater than that of privately-owned automobiles. In Michigan the reverse is true. The committee will attempt to determine the proper contribution.

L. E. Conrad of Kansas State Agricultural College delivered his report on wind resistance to motor vehicles. In summarizing available information on the subject, he pointed out that only three investigations are known to have been made in the United States, those of: (1) The United States Bureau of Standards; (2) the engineering experiment station of Kansas Agricultural College; and (3) the engineering experiment station of Iowa State College. Some work has also been done along similar lines in Germany.

Professor Conrad concludes that for a straight head wind the resistance for ordinary passenger cars may be considered as $0.0025 V^2$ pounds per square foot of projected area. In this formula V is equal to the velocity of the automobile in miles per hour. He believes that an even exponent of V will serve as well as a fractional one for all practical purposes. Studies are to be continued on the effect of side and quartering wind.

H. B. Shaw, of North Carolina State College, gave a résumé of research on tractive resistance. An attempt is being made to formulate a practical theory based upon the results of outstanding studies. As an interesting side light he recalled that Sir Isaac Newton, years ago, developed theoretically a value for air resistance that has since been confirmed experimentally.

The present experiments indicate that tire displacement resistance decreases with increased air pressure in the tire which results from the heating which takes place when a vehicle is in motion. Tire displacement has been found to increase at a greater rate than the increase of the weight on the tire.

MANY PHASES OF STRUCTURAL DESIGN OF ROADS STUDIED

A. T. Goldbeck, of the National Crushed Stone Association, chairman of the committee on structural design of roads, because of the limited time, summarized the reports of the members of his committee. These papers covered 19 subjects. With regard to the loads on highways the speaker stated that with the same total load the rear wheels of four-wheel trucks transmit twice as much load to the pavement as either pair of rear wheels of the six-wheel truck.

The report on subgrade studies outlined methods of soil identification from known proportions of the separate of sand, silt, and clay. The slaking value test was mentioned as indicating the susceptibility of soil to erosion. A simple percolation test was considered to be adequate to determine those soils which might be drained with tile. The report continued with a discussion of the behavior of subgrade soils when subjected to various moisture conditions. Granular sub-bases were mentioned as an efficacious method of compensating undesirable subgrades. The studies of landslides, and the sand-clay, gravel, and top soil surfaces of the Southern States were outlined.

A rough practical rule given for the construction of bituminous macadam surfaces was that the depth of the surface course in inches should be equal to the maximum size of stone in the surface course, and the number of gallons of bitumen per square yard of surface might be estimated at the same numerical figure. Thus for a $1\frac{1}{2}$ -inch surface course, the maximum size of stone would be $1\frac{1}{2}$ inches, and there would be required $1\frac{1}{2}$ gallons of bitumen per square yard.

With respect to the design of concrete pavements, the speaker stated that this is affected by the character and condition of the subgrade, and that the latter must be evaluated in order to make possible a more accurate cross section. The maximum wheel load is now recognized as causing the stress for which the pavement must be designed. There is a noticeable tendency on the part of highway engineers to reduce the size of concrete pavement slabs with longitudinal and transverse joints.

Mr. Goldbeck concluded his summary with comments concerning reinforcing and curing of concrete pavements, and methods of constructing brick pavements.

In a general discussion of the report of the committee on structural design of roads, C. H. Moorefield, State highway engineer of South Carolina, stressed the importance of considering the subgrade and emphasized the need of further knowledge of subgrade soils and their behavior as a prerequisite to adequate pavement design.

REPORT OF COMMITTEE ON CHARACTER AND USE OF ROAD MATERIALS

The report of the committee on character and use of road materials was presented by the chairman—H. S. Mattimore of the Pennsylvania State highway department. The paper discussed six factors in the control of the construction of concrete pavements: (1) Retesting of cement stored over three months; (2) fine

aggregate; (3) coarse aggregate; (4) proportioning of materials; (5) curing the finished pavement; and (6) tests of the finished pavement.

The quantity of Portland cement used, said the speaker, should be carefully checked at the plant, on the job, and by counting the empty sacks. With regard to fine aggregates, uniformity, durability, and a uniform degree of hardness and toughness, are desirable. Stone screenings contain too much dust for satisfactory use as concrete aggregate. They also make it more difficult to finish the pavement. Seven theories of designing concrete mixtures were discussed.

In commenting on the recently developed methods for curing concrete pavements, using such materials as calcium chloride, sodium silicate, and bitumen as a seal coat, Mr. Mattimore stated that all of these, from an economic standpoint, are superior to the customary water-curing method. The new methods, however, have a number of disadvantages in their practical application. Reports indicate that calcium chloride may not be desirable for surface curing because of excessive scaling of the pavement surface. As an admixture the success of calcium chloride depends on atmospheric conditions. In the Northern States, under average conditions of temperature and humidity, it is possible to obtain good average concrete with a calcium chloride admixture. In the successful use of seal coats, the character of the subgrade, whether porous or impervious is an important factor.

Discussing tests of finished pavements, Mr. Mattimore stated that three methods are in general use: (1) A compression test on specimens made and partially cured in the field; (2) a compression test on cores drilled from the finished pavement; and (3) a transverse beam test on specimens made and cured in the field. Although the prevailing practice has been to keep concrete pavements closed for three weeks, the indications are that it is better practice to vary the time according to the results of tests. Up to the present time the procedure for making the field transverse test has never been standardized so as to produce comparable results by different observers.

P. J. Freeman of the Allegheny County department of public works, in Pennsylvania, discussed the practical aspects of Mr. Mattimore's report. He believed that not enough attention has been paid to the weight of the bagged cement at the mill. In some cases observations show a difference of 13 per cent between the high and low weight. Careful consideration should be given the proposed plan of proportioning the cement by weight. With regard to the bulking of sand, it is not a difficult matter to determine the percentage of moisture in the field. The weighing and inundation methods seemed to be the simplest yet devised.

Referring to the committee's recommendations relative to the sampling of coarse aggregate, he differentiated between the purposes of the sampling. If it is desired to determine the average quality of coarse aggregate, say on a barge, then the suggested method is satisfactory, but where it is desired to determine whether one end of a pile is badly segregated, then a method other than that proposed by the committee would have to be used.

With reference to sampling, proportioning and design of concrete mixtures, Mr. Freeman believes that more attention should be given to prevention of segregation of aggregate. To accomplish this, flat rather than conical piles should be used.

Referring to the proposed method of proportioning concrete by combining definite weights of the several constituent materials, the speaker believes that nothing more elaborate is being advocated than what has been in use for years in the construction of bituminous pavements.

Mr. Freeman emphasized the urgency of placing wet burlap on the concrete as soon as possible for curing. He fears that too much time is being spent on finishing. Stating also that excessive checking is caused by too early belting, he believes there is a happy medium of time and that more research is needed concerning this factor.

Calling attention to the present methods of testing the finished pavement, he stated his belief that it is illogical to close pavements for a standard time of three weeks in July and November, because of the widely different weather conditions. Some of the concrete pavements in Allegheny County, laid in the summer, have been opened successfully in three days, while other projects, built late in the fall, have not been opened until the following spring. For the traffic in Allegheny County Mr. Freeman believes that a modulus of rupture of 500 pounds per square inch, as determined by the beam test, gives an ample factor of safety.

D. A. Abrams, acting as presiding officer at the morning session on December 2, introduced A. J. Brosseau, who spoke on highway finance. The speaker analyzed the aggregate Federal expenditures and showed the relation thereto of the Federal highway disbursements. The Federal road expenditures were shown to be less than 8 per cent of the entire rural highway bill. Mr. Brosseau believes that our national highway finance structure is sound, necessary, and profitable.

In discussing Mr. Brosseau's paper, Professor Agg called attention to a new problem that is arising in highway finance. His studies indicate that, in some instances, municipalities are requesting that a portion of the gasoline tax be prorated to them for the construction and maintenance of the city streets. The city argument is that their motor-vehicle users are paying for a large part of the cost of rural roads, for which they are not receiving an equitable return. Professor Agg pointed out that if the gasoline taxes are returned to the cities the tendency will be to dissipate the funds which are now being used through a central State agency to secure maximum results on the rural roads. In discussing this point, J. G. McKay, of the Bureau of Public Roads, presented some conclusive arguments. He stated that traffic surveys of the bureau indicate that 80 to 90 per cent of the rural-road traffic and the bulk of the heavy-truck traffic originate within city limits. It is, therefore, logical that the city motor-vehicle owners should bear the bulk of the cost of the rural roads. He also added that there is no reason why the rural land owner should pay for any considerable portion of the through trunk roads because land values in the strictly rural areas are not increased to a large extent by this kind of road development.

REPORT OF COMMITTEE ON HIGHWAY TRAFFIC

G. E. Hamlin, of the Connecticut State Highway Commission, as chairman, introduced the individual reports of the committee on highway traffic. He stated that highways should be classified with respect to their weight-carrying capacity, and that there is an urgent

need for establishing adequate rights of way in congested rural areas. Computations were presented showing that the maximum capacity of a single-lane road amounting to 1,969 motor vehicles per hour occurs at a speed of 22 miles. The maximum capacity of a two-lane road with mixed speeds is somewhat less than twice that of the single-lane road. He outlined the effect of overcrowding a two-lane road and showed the magnitude of the time losses at grade crossings and in small villages. The precautionary stop of a motor vehicle at a grade crossing was said to average seven and one-half to eight seconds. A comparison was made of the time consumed on by-pass or belt routes around centers of population, as contrasted with through roads. Forecasting the future development of traffic the speaker saw an increase in the use of six-wheel trucks.

J. G. McKay, of the Bureau of Public Roads, as a member of the committee, gave a description of the new traffic-flow recording device now being used by the bureau in the traffic studies in the Cleveland metropolitan area. An attempt is being made to determine the traffic capacity of various widths of roads as well as the preferential speed for various densities of traffic.

A. N. Johnson showed lantern slides of the airplane pictures taken during the traffic study on the Washington-Baltimore boulevard. The pictures were taken at an elevation of 3,600 feet and showed a strip of land 2,000 feet on each side of the road.

H. J. Kirk of the Ohio State Highway Department gave a description of a traffic speed recorder which had recently been manufactured by a firm in Dayton, Ohio. The variations in speed of a motor vehicle are shown by variation in the slope of a graph made upon paper mounted upon revolving cylinders. The device has been used on delivery wagons to determine if drivers are making the rounds on a regular schedule and may be useful in traffic flow studies.

Discussing the report of the committee on highway traffic, W. G. Sloan, State highway engineer of New Jersey, stated that traffic studies are vital to the solution of the highway transportation problem. Basing his remarks upon traffic surveys made in New Jersey, he estimates a 200 per cent increase in traffic in 1932, as compared with 1923. Traffic surveys and gasoline consumed indicate that the annual mileage traveled by automobiles is increasing more rapidly than the motor-vehicle registration. A graph prepared by the New Jersey department shows 1,600 vehicles as the maximum capacity of a single-traffic lane at 22 miles per hour, while the curve of the committee indicates 1,969 vehicles. The speaker emphasized the economic loss caused by the interruption to the flow of traffic at grade crossings in addition to the loss due to accidents and fatalities.

SELLING RESEARCH TO THE PUBLIC

Maurice Holland, director of the division of engineering and industrial research, gave an address on the subject of selling research to the public, or, as he expressed it, making the public research conscious. There are at present 1,000 research laboratories in the country and this is an increase of almost 100 per cent in six years. Stating that a man's success in business is in direct proportion to his ability to sell his services, he believes that until research can be translated into the language of the man on the street, we can not expect to obtain general attention. Lindbergh's flight was

an example of the popularization of science, and to-day, as a result, 117 airplane factories in this country are running behind their production schedule.

The speaker said that the results of research must be made apparent and that proper showmanship is necessary to popularize the message. He referred to Pasteur as a master showman in reporting his studies to the French Academy, before whom he demonstrated the effects of living anthrax germs.

Speaking of the four mediums for reaching the public, he said that with the spoken word, in lecture form, only the attention of a few hundred people at best could be held. With articles written in trade journals 6,000 to 60,000 people could be reached. He mentioned the motion picture as an untried medium from a research standpoint, and added that one weekly news reel reaches 12,000,000. The speaker believes the radio the least expensive and most effective means of disseminating public information. One broadcasting station in New York City reaches four or five million people nightly.

CULVERT INVESTIGATION REPORTED

R. W. Crum, of Iowa State College, as chairman, presented the report of the committee on culvert investigations. Little additional field work had been done during the past year, but study had been made of previously collected data. Two general classes of culverts were considered: The rigid, such as concrete, vitrified tile, and cast iron; and the flexible, such as corrugated metal. It is difficult to rate the relative condition of rigid-type culverts since progressive deterioration is not indicated. With corrugated metal, however, the progressive deterioration could be determined. The committee, therefore, suggests a rating method for corrugated-iron culverts. In this rating, three stages of deterioration in the ordinary life of culverts are recognized: (1) Invisible, (2) visible, and (3) unsafe. The end point of the life of the culvert is used for comparison, and this is based upon the deterioration of the metal in the culvert.

W. H. Root, of the Iowa State Highway Commission, chairman of the committee on maintenance, presented a summary of the studies made of recent developments in road maintenance. C. P. Owens, of the Missouri State Highway Commission, led the general discussion of the paper which closed the morning session.

PROGRESS MADE IN LOW-COST ROAD INVESTIGATION

At the afternoon session C. N. Conner, chairman of the committee on low-cost road improvement, gave a detailed study of the construction practices determined by field inspections in 23 States. The work of the committee has been financed by the American Road Builders' Association and T. Coleman du Pont. The purpose of the study is to assist those engaged in the construction of this type of road. Studies of typical cross-sections in 26 States were shown with lantern slides. The low-cost roads studied have an average traffic of 600 and a maximum of 1,500 per day. The survey has been limited to untreated surfaces costing less than \$10,000 a mile and surface treatments costing less than \$6,000 a mile. The report presented was a digest of the full report which contains 60,000 words.

H. J. Kirk, of the Ohio State Highway Department, followed with a discussion of untreated traffic-bound types of low-cost roads.

C. A. Hogentogler, of the Bureau of Public Roads, presented a report on subgrades and soils, indicating their relation to the low-cost road problem. The subgrade soil observer attempts to determine where rigid or flexible road surfaces may be laid. The surest subgrade treatment was believed to consist of a granular subbase compacted under traffic.

N. S. Anderson, of the South Carolina State Highway Department, described the surface treatment of main highways with topsoil and selected earth. For bituminous surface treatment, the base should be sufficiently porous to permit the tar prime coat to penetrate one-fourth inch. Excessive mica in the material makes the base unstable.

W. R. Neel, of the Georgia State Highway Department, described untreated surfaces of sand-clay, chert, and gravel. He discussed the materials, methods of construction, cost of construction and maintenance, serviceability and salvage value. The average cost of sand-clay roads in Georgia is \$1,650 a mile and the average annual maintenance costs \$170 per mile. These roads are serviceable for 400 vehicles a day or less. For this amount of traffic the annual loss of surfacing material is one inch of depth. The sand-clay road is of great value as the first step in stage construction for stabilizing subgrades.

Chert construction costs up to \$7,000 a mile and carries up to 1,000 vehicles a day. The average annual loss in depth is one-half inch.

On gravel roads, the traffic should not exceed 1,000 vehicles a day. The average annual maintenance cost is \$200 a mile. The average annual loss in depth is three-fourths of an inch.

The Georgia State Highway Department has co-operated with Doctor Strahan, of the University of Georgia, in an extensive study of low-cost roads which has recently been completed.

J. T. Pauls, of the Bureau of Public Roads, presented a report on mixed-in-place surfaces, using local topsoil and gravel aggregates and tar or asphalt. The maintenance costs of untreated roads are excessive for a traffic greater than four to five hundred vehicles a day and resort is being had to various bituminous materials. Preliminary studies have been carried on in several States. To obtain good results with this type, correct maintenance is as necessary as first-class construction.

V. R. Burton, of Michigan, presented a paper on mixed-in-place surfaces of stone or gravel and fuel oil, prepared by C. L. McKesson, of the California State Highway Commission. This paper was in the nature of a discussion of Mr. Conner's paper. Mr. McKesson believes that untreated rock and gravel surfaces can not be justifiably termed low-cost roads when all the dependent costs are considered, such as maintenance, replacement, interest on first cost, excessive tire wear, motor-vehicle depreciation, and gasoline expenditures. Bituminous treatments according to recent California and Oregon practice are believed to so reduce these costs that the resulting surface might be classed as a low-cost road. Mr. McKesson had just returned from a trip abroad and it is his impression that European highway engineers agree that untreated gravel or stone roads are no longer economical.

B. E. Gray, of the West Virginia State Road Commission, illustrated his paper on the use of soft stone in the construction of puddle macadam with a number of lantern slides. The construction of the soft sandstone surfaces was said to range in cost between waterbound and penetration macadam. The annual maintenance costs, for traffic not exceeding 800 vehicles per day, does not exceed \$200 to \$300 per mile.

W. A. Van Duzer, of the Pennsylvania State Highway Department, discussed maintenance methods and equipment, referring to the maintenance organization of his State.

J. G. McKay, chief of the division of highway transportation and economics of the Bureau of Public Roads, presented a paper on traffic capacity and service of low-cost roads. He stated that the life costs of low, middle, and high-class surfaces must be determined before an intelligent selection of types may be made. Referring to the large mileages of low-type roads in the country, he stated that traffic studies of the bureau indicate that even in the densely populated State of Ohio, 70 per cent of the State highway system carries less than 600 vehicles a day. In both Pennsylvania and Vermont, 70 per cent of the primary road systems show an average daily traffic of 600 vehicles or less. With regard to the maximum wheel loads of trucks, the speaker believes that a 7,500-pound load per wheel will cover the maximum loading on 75 per cent of the State systems. Less than 1 per cent of the Pennsylvania State highway system carries wheel loads in excess of 9,000 pounds.

EFFECT OF SALTS IN MIXING WATER ON COMPRESSIVE STRENGTH OF MORTAR

AN investigation has recently been completed at the University of Texas on the effect of various salts in the mixing water on the compressive strength of mortars.¹ In many parts of Texas, the only waters available for use in mixing concrete contain relatively large percentages of soluble salts, the so-called "gyp" and "alkali" waters being frequently encountered. It has generally been considered that the sulphates are the most injurious salts, and tests of such waters usually have consisted in a determination of the effect of the water on mortar strength in comparison with a water of acceptable quality, with occasional tests of total solids and percentage of sulphate ion. However, since it was not known definitely what percentage of sulphates might be expected to be

injurious, a chemical analysis furnished little information of value.

To obtain definite quantitative results on the effect of some salts which might be present in mixing waters, a preliminary series of tests was started in 1922. These tests consisted of the determination of the compressive strength of 480 2-by-4-inch cylinders of neat cement mixed to normal consistency, using distilled water with the addition of various percentages of sodium chloride, sodium sulphate, sodium carbonate, magnesium chloride, and magnesium sulphate. These tests were made at ages of 28 days, 3 months, 1 year, and 3 years.

After these tests were started it was thought advisable to make similar tests on 1:3 (by weight) sand mortars, and to include some salts not used in the preliminary tests. In this series of mortars the following salts were used: Sodium chloride, sodium sulphate,

¹ University of Texas Bulletin No. 2730, Effect of Various Salts in the Mixing Water on Compressive Strength of Mortars, by F. E. Giesecke, H. R. Thomas, and G. A. Parkinson.

sodium carbonate, magnesium chloride, magnesium sulphate, calcium chloride, and ferrous sulphate. These tests were made on 2-by-4-inch cylinders, which were tested at the same ages as the neat cement.

In each case the amount of salts used was based on desired percentage of negative ion in the mixing water, the concentrations being 0, $\frac{1}{2}$, 1, 2, and 4 per cent.

In the main group of 1:3 mortars consisting of 700 cylinders, the sand used was from the Colorado River at Austin. In another smaller group consisting of 500 cylinders, a natural limestone sand from Burnet County, Tex., having practically the same grading as the Colorado River sand, was used. In this latter group the calcium chloride and ferrous sulphate were omitted.

Materials.—(1) Cement: The cement used was a laboratory blend, made up of a mixture of two brands of Texas Portland cement. This blend met all of the requirements of the specifications and tests of Portland cement of the American Society for Testing Materials. (2) Sand: The Colorado River sand consists essentially of quartz and feldspar grains, with a small percentage of limestone grains. The natural limestone sand consists of clean grains of a good quality of limestone, having practically the same grading as the Colorado River sand. (3) Water: In all of the mixes distilled water was used, with the amounts of salts added as required for the several groups. The chemical formulas on which were based the quantities of salts required to give the desired percentages of the various negative ions are as follows:

Material	Formula	Molecular weight	
		Total salt	Negative ion
Sodium chloride.....	NaCl.....	58.5	35.5
Sodium sulphate.....	Na ₂ SO ₄ , 10H ₂ O....	322.2	96.1
Sodium carbonate.....	Na ₂ CO ₃	106.0	60.0
Magnesium chloride.....	MgCl ₂ , 6H ₂ O.....	203.3	71.0
Magnesium sulphate.....	MgSO ₄ , 7H ₂ O.....	246.5	96.1
Calcium chloride.....	CaCl ₂	111.1	71.0
Ferrous sulphate.....	FeSO ₄ , 7H ₂ O.....	277.9	96.1

Fabrication of specimens.—For each group of cylinders having the same percentage of added salt, five mixes were made on five different days in order to minimize accidental variations. Each mix was sufficient to make four 2-by-4-inch cylinders, which were tested at the four ages. The materials for each mix were accurately weighed in the proportion of 1 part cement to 3 parts dry sand. The amount of water for use with each batch was measured and the calculated weight of salt thoroughly dissolved in it, when it was used in the mix. The consistency of the mortars was as nearly as possible the same as the standard consistency used in cement testing. The mixing and molding of the specimens were carried out as recommended by Committee C9-16T of the A. S. T. M., except for finish of ends.

Storage.—After molding, the cylinders were placed in the damp closet for approximately 24 hours, after which they were removed from the molds, marked for identification, and placed in running water in a storage tank where they were kept until time for testing.

Capping and testing.—A few hours before testing the cylinders were removed from water and capped with plaster of Paris. The specimens were tested in a 100,000-pound capacity, screw-type testing machine, a 2-inch spherical bearing being placed on top.

The results of the various series of tests are given in the report on the investigation and comparisons made on the basis of the ratio of the unit strength at a given age for a given percentage of added salt to the strength for no added salt. They show that all of the sodium salts used are injurious to Portland-cement mortars,

the chloride, sulphate, and carbonate showing progressively greater reduction in strength for a given percentage of negative ion. The two magnesium salts used have only slight effect. Calcium chloride and ferrous sulphate are beneficial.

It is quite evident from the results that, so far as mixing water is concerned, the sulphate ion is not necessarily injurious to the strength of Portland-cement mortars. Of the three sulphates used, the sodium salt is injurious, the magnesium salt shows slight effect, and the ferrous sulphate increases the strength materially.

Compared to the percentages of salts used in these tests, the waters from springs, streams, and wells will, in general, contain relatively small percentages of dissolved salts. From a study of the chemical analysis of waters from many parts of the State, it is noted that in the great majority of cases the total solids are less than 5,000 parts per 1,000,000, or about one-half of 1 per cent. Even if all of this amount of salt were sodium carbonate (which was the most injurious of the salts tested) this would correspond to about 0.3 per cent of carbonate ion, which probably would not reduce the mortar strength more than about 5 per cent.

In presenting conclusions as to the results of the tests, attention is called to the fact that the results given apply only to salts present in the mixing water and that all brands of cement may not be affected in the same way. The following conclusions are presented as a general summary of the results obtained.

(1) Sodium salts (chloride, sulphate, and carbonate) are injurious to Portland-cement mortars.

(2) Magnesium chloride and sulphate have very little effect on mortar strength.

(3) In general, the strength ratios tend to increase with age—that is, for a salt that reduces the strength, the reduction is less for greater ages, and for salt that increases the strength the percentage increase at 3 years is usually greater than at 28 days.

(4) Sulphates are not necessarily injurious to mortar strength.

(5) Two per cent of sulphate ion in the form of ferrous sulphate—that is, about 6 per cent of the salt—increases the mortar strength approximately 20 per cent.

(6) Relatively few natural waters contain high enough percentages of total solids to make them unsafe for use in concrete.

While it is probable that ferrous sulphate will not be present in natural water, it was included in this series in order to determine the effect of an iron salt on the strength of Portland-cement mortar. For some years it has been noted by members of the laboratory staff that natural sands containing finely divided iron oxide seemed to show abnormally high strengths, and it has been the opinion that the iron present might have some chemical reaction with the cement, thus increasing the mortar strength. The results obtained with this salt seem to confirm the opinion. It would be interesting to know the effect of some other iron salts, and it is believed that it would be worth while to extend this part of the investigation to include the effect of various commercially available iron salts on the strength and other properties of Portland-cement mortars and concrete.

ELEMENTARY PROOF OF SHALE-LIKENESS OF CLAY PARTICLES

By DMITRY P. KRYNINE, Professor of Highway Engineering, Moscow Superior Technical School, and Moscow Institute of Transportation Engineering

AS IS well known, one of the essential differences between sand and clay lies in the dimensions and shape of particles of both soils. Sand particles have the shape of more or less large seed, and fine clay particles are shaped like shale or plates. This difference may not be established directly because of the small dimensions of clay particles which can not be isolated by any of our laboratory instruments for the purpose of studying them by the aid of a microscope, and in many cases the microscope is useless as our eye is unable to discern very small particles.

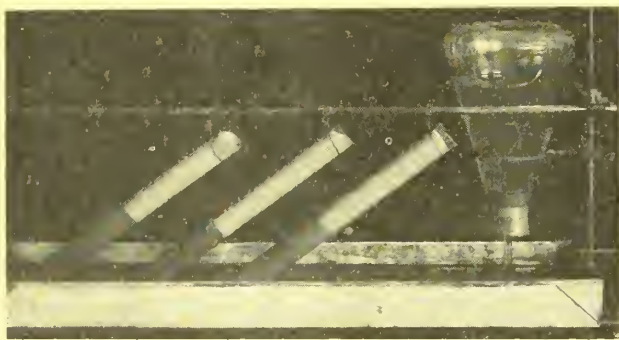
A proof of the shale-likeness of clay particles is given in Doctor Terzaghi's well-known book *Erdbaumechnik*.¹ In the experiment described by Doctor Terzaghi, water containing suspended clay particles $2\ \mu$ in size ($1\ \mu = 0.001$ millimeter) remained between two

as in the sand. With sand the level of the rising water is parallel to the surface of the water in the basin; i. e., it is horizontal. The pores of the sand seem to be disposed in disorder, the way of the water movement is relatively straight, and the capillary water rising to the surface passes approximately in the same direction through all the pores.

Now take a tube filled with dry clay powder; naturally the bottom end of the tube must be previously covered with cheesecloth. Before beginning the experiment clay particles and balls must be carefully broken into powder, and the powder strewn into the glass tube slowly and with great accuracy (for instance, from a constant height of 1 meter and through a rubber tube, as it was done in the laboratory mentioned below). This phenomenon is similar to that which took place in nature many centuries ago when the sedimentation of many contemporary clays was effected in the water. During the latter proceeding the clay particles probably disposed their broad side approximately horizontally on the lower layers, one layer being separated from another by a thin horizontal cleft. The tube is jarred until the soil reaches a certain height and no further settlement takes place. Then the glass tube is put into an inclined position and its lower end immersed in the water. The surface of contact between the soil and the water may be inclined or horizontal. In both cases the water rises as shown in the photograph; i. e., its surface is not parallel to the surface of the water in the basin but normal to the axis of the tube. This can only be explained by the shale-like shape of the clay particles. When the tube is placed in an inclined position the clefts between the layers also become inclined. The water rises through them and seeks a passage upward. This passage is found between the particles of the lower layer, then the water fills the following cleft, and so on.

The device at the right of the photograph has no special influence on the results of the experiment. It is merely an overturned cucurbit with water to maintain the water in the basin at a permanent level. A thin glass tube passes through its cork, the bottom end of the tube having a sloping cut. At the lowering of the level in the basin the air enters the cucurbit and releases a correspondent quantity of water to replace the loss.

The simple fact above described seems to be an undoubted proof of the shale-likeness of clay particles. It was observed in the highway laboratory of the Moscow Superior Technical School. (Prof. D. P. Krynine; Miss Mary V. Ivanova and Mr. T. A. Ovssiannikoff, both graduates in civil engineering, coworkers.)



DEMONSTRATION OF RISE OF CAPILLARY WATER IN CLAY

watch-glasses located at a distance of $0.1\ \mu$ from each other, this distance being established by the color of Newton rings surrounding the water spots between the glasses. The phenomenon may only take place if clay particles are shale-like.

A practical highway engineer is not able to reproduce this experiment owing to lack of necessary laboratory equipment. An elementary proof of the shale-likeness of clay particles is given below and the suggested experiment can be easily repeated by anyone.

Everybody knows that if the lower end of a tube filled with dry soil is immersed in water the rise of water due to capillary action begins. If the tube is held vertically it is quite natural for the level of the water rising uniformly in the capillary pores of the soil to be normal to the axis of the tube. Logically it must be so. If the tube is inclined the case is different. In this case the experiment shows that the rise of capillary water does not take place in the same way in the clay

¹ Published in Vienna, 1925.

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UNITED STATES DEPARTMENT OF AGRICULTURE
BUREAU OF PUBLIC ROADS



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FEBRUARY, 1928



STUDIES OF POWER-SHOVEL OPERATION SHOW POSSIBILITIES OF INCREASED EFFICIENCY

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U. S. DEPARTMENT OF AGRICULTURE

BUREAU OF PUBLIC ROADS

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R. E. ROYALL, Editor

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POWER-SHOVEL OPERATION IN HIGHWAY GRADING

A REPORT OF OBSERVATIONS MADE ON GOING PROJECTS BY THE DIVISION OF MANAGEMENT,
BUREAU OF PUBLIC ROADS

Reported by T. WARREN ALLEN, Chief, Division of Management, and ANDREW P. ANDERSON, Associate Highway Engineer

PART 1.—AN OUTLINE OF MORE IMPORTANT FACTS DEVELOPED IN STUDIES AND DISCUSSION OF FACTORS AFFECTING THE OPERATING CYCLE OF THE SHOVEL

THE DAILY cost of operating a power-shovel grading outfit is very nearly fixed for any given set of conditions, regardless of whether the output is high or low. The only effective means available to the contractor for reducing his unit cost is therefore to increase the rate of production. Some of the more general requirements necessary for efficient, economical production as developed will be briefly summarized in the first portion of this article and treated in greater detail in subsequent portions of this series of articles.

Efficient use of the power shovel in highway grading generally involves the proper coordination of at least three distinct operations: (1) Except where the material can be cast, it must be dug and loaded into the hauling units at or near the maximum rate of production for the material handled; (2) the hauling units must be just sufficient to carry the output of the shovel and must be operated with almost clocklike precision, so that the load may be received, transported to the place of disposal, dumped, and the hauling units returned to the shovel without interrupting the steady operation of the shovel; (3) at the fill or dump the material brought by the hauling units must be spread and compacted or otherwise cared for as may be required by the specifications without interfering with the steady operation of the hauling units. If the material is too hard to be dug effectively with the shovel, another operation, that of drilling and blasting, is necessary, and this operation must also be carried on without interfering with the others.

Efficient operation requires not only that a high rate of production be secured with the shovel, but that this production be secured with the use of a minimum of labor and auxiliary equipment. This can only be accomplished when each operation is so synchronized and coordinated that the entire organization functions as a unit without either interference or waste in any of

its parts. Absolute perfection in all details is probably impossible. Nevertheless, recent extensive studies by the Division of Management of the Bureau of Public Roads, of power-shovel operation on a large number of projects operated under a great variety of conditions show rather conclusively, (1) that a high degree of efficiency is possible of attainment, and (2) that, in general, the cause of the low production encountered on many projects is due to conditions over which the management has more or less definite control and which are therefore to some extent remediable.

On this work the shovel is the primary producer. All production is dependent on it. An inferior shovel or operator is a certain guarantee that production costs will be high. The shovel should be sturdy, powerful, dependable, fast, and easily operated. But no matter how good the shovel, a high grade of skill, intelligence, and endurance is required on the part of the operator in order to secure consistently a high rate of production.

In ordinary common excavation 4 or more feet in depth and which is dug easily and dumped freely, a good power shovel in good condition can load vehicles at the rate of four dipper loads per minute, providing the vehicles are so placed that the average swing does not exceed 90°. A good operator can continue this rate for intermittent periods throughout the day. To attain this rate it is necessary to load the dipper in about 4½ seconds, to swing and spot the dipper in about 4 seconds, to dump it in 1½ seconds, and then return the dipper to the loading point in about 5 seconds. Many jobs have been found where this rate has been maintained during intermittent periods of varying length under the conditions given above, and it may therefore be taken as the maximum attainable with present-day power shovels worked under favorable field conditions. However, numerous jobs have

DATA COLLECTED in studies of power-shovel operation, which are reported in this series of articles, indicate that there are few jobs on which a material increase in output can not be obtained without a corresponding increase in cost. In some cases it has been found possible to increase output as much as 100 per cent by a change in management methods.

Under favorable conditions a good operator can attain a rate of loading of four dipper loads per minute for intermittent periods and three dipper loads or more per minute as a continuous rate. By whatever amount the contractor fails to attain this rate under favorable conditions, he is failing to attain the maximum possible efficiency.

Efficiency of production can not exceed the efficiency of the shovel operator. To change an operator making a load every 18 seconds for one who takes 20 seconds reduces the output 10 per cent. This can easily amount to \$20 to \$25 a day in lost profits.

The difference in time required to make a 90° swing in loading and to make a 180° swing may reduce the output by as much as 25 per cent.

A mass diagram is essential in planning hauling equipment and estimating on a job. With such a diagram, and knowing the time constants of operation, it is possible to determine the most efficient outfit and how long it should take to do the work.

Determination of proper team supply greatly affects profits. One job is estimated to cost \$15,440 with an 8-team outfit which can be done for \$14,555 with an 11-team outfit.

It is often advantageous to sublet short-haul work or to do it with wheelers or frescoes when conditions result in a surplus of teams at the shovel.

Work should be continued under adverse conditions when it is possible to earn more than the difference between the costs which accrue while working and those which accrue while idling.

It has been found that the diversity of kinds and sizes of trucks has produced varied results in meeting the specialized requirements of use with a power shovel.

Using trucks on a short haul, it has sometimes been found possible to double the output by backing them to the dump rather than making a turn at the dump and another at the shovel.

Roadway conditions play an important part in truck operation. Pneumatic tires, particularly those of the dual type, are preferable.

Crawler-type tractors drawing large dump wagons have been found adapted to conditions commonly found on power-shovel jobs. The number of trains used must agree closely with economical requirements. As an example, the use of one 2-wagon train on a particular job requiring two such trains would have increased the cost of the job nearly \$7,200, while the use of three trains would have increased it \$1,250.

been found where the average rate of all-day shovel operation, in good common, was at the rate of three or more dipper loads per minute, and this may therefore be accepted as a criterion of good operation under normally favorable field conditions. If the operator is forced to swing his shovel 180° instead of 90° , his best possible short time output will be only about $3\frac{1}{6}$ dipper loads per minute even with a very fast swinging shovel, while his all-day average rate may readily be much less than $2\frac{1}{2}$ dipper loads per minute.

Output, however, is the product of the number of dipper loads multiplied by the average yardage per



A FULL DIPPER LOAD OF EASILY WORKED MATERIAL AND A SHORT SWING. SUCH CONDITION SHOULD PRODUCE A LARGE OUTPUT

dipper load. A good operator can combine both speed and high average quantity of material. In ordinary common excavation 3 or more feet in depth the average dipper load for a $\frac{3}{4}$ -yard shovel should be about one-half cubic yard of material as measured in place. A 1-yard dipper should average about 0.7 cubic yard. In some materials which heap up on the dipper and do not spill on the swing the average load will sometimes equal the rated capacity. In poorly blasted rock or shale, or in material full of roots and stumps, the average dipper load may be 40 per cent less than the general average for ordinary common excavation or about 0.3 cubic yard for a $\frac{3}{4}$ -yard dipper. Figure 1 is illustrative of the studies made and shows the rate at which dipper loads can be deposited in the hauling units under fast operation, and how a few slow operations increase the average time per cycle for the entire period. As the material passes from good common to one more difficult the digging and loading operations become slower and there is greater difficulty in securing a full dipper.

MISTAKES IN MANAGEMENT FOUND WHICH GREATLY REDUCE PROFITS

The custom of loading the hauling units at the rear of the shovel (180° swing) is a very expensive practice. Even with a fast-swinging shovel, loading at the rear of the shovel instead of at the side will increase the time required for each dipper load about four seconds, and if the shovel is of slow-swing speed it may be twice this amount. If the average time per dipper load is 20 seconds when loading at the side of the shovel, it will be somewhere between 24 and 28 seconds if the loading is at the rear. In other words, production will be cut from 180 dipper loads per hour to 150 or possibly as low as 128 per hour. Consequently, one of the essential requirements for high-shovel production is to so place the wagons or trucks that the swing of the shovel will be as short as possible.

For materials which clear the dipper freely the dumping time should not, in general, exceed an average of about one second, but sticky, adhesive materials require much skill on the part of the operator, if the average dumping time is to be held down to two or three seconds. A slow or inexperienced operator may readily consume two or three times as much time per dipper load. Daily production in very sticky or adhesive materials may therefore be as low as for poorly blasted rock.

The general tendency among highway grading contractors seems to be to do too little blasting as well as too little clearing and grubbing. It is not uncommon to find power shovels producing only about one-half the output which would have been possible had the blasting been well done. In materials which are too hard to dig readily without blasting, a good powder man is indispensable for satisfactory production.

In shallow cuts attention should be given to the time required to move the shovel forward. A modern crawler-type shovel in good condition can be moved forward in 15 seconds. If an average of 30 seconds is required, either the operator is slow or else the mechanism needs attention. The old-style wheel-traction type operating on mats will ordinarily require from five to seven times as long to move. This is a severe handicap where much shallow cutting is involved. In shallow cuts keeping the boom lower than normal and the shovel well forward will facilitate filling the dipper, while in deep cuts the boom should be kept high and the shovel well back from the face.

On the jobs studied an inadequate supply or poor operation of hauling equipment, or both, were the most frequent causes of slow-shovel operation in good common excavation. Trimming to grade and dressing slopes was the second most prolific cause for extending the time required per dipper load.

If the highest possible shovel production is to be secured, vehicles must be exchanged within the time required to handle one dipper load, or in good common from about 12 to 18 seconds. To operate the hauling equipment so as to meet this requirement is practical under ordinary field conditions, providing each vehicle can carry two or more dipper loads per load. It is not possible to so synchronize the operation of the hauling equipment that high production can be maintained consistently if only one dipperful is carried per load.

NUMBER OF HAULING UNITS NEEDED REQUIRES CAREFUL CONSIDERATION

Since the shovel can, in general, only dig material when vehicles are in position to receive it, the adequacy of the hauling equipment has a very decided effect on production. The number of hauling units of any given kind which are required varies almost directly as the length of haul which generally varies between rather wide limits and often at rapid, irregular rates. The characteristics which affect the rate at which the material can be dug by the shovel sometimes also change with unexpected frequency. In practice, therefore, it is found inadvisable to attempt to maintain an exact balance between the hauling equipment supplied and that just needed to maintain the highest rate of production at the shovel. Consequently, in many instances a definite number of hauling units is sent out and maintained on the job until it is complete. The result is that on very short hauls some of the equipment is idle or working at slow rate, while on the longer hauls not enough equipment is available to keep

the shovel at a high rate of production. The question thus becomes one of determining what hauling equipment should be sent out in order to complete the job at the lowest possible cost.

Since it is general practice to maintain a fairly definite number of hauling units on the job, the utilization of the otherwise idle time on the short hauls therefore becomes a matter of considerable importance. At present, the general practice is to begin the fill at the nearest point of the cut, so that as the work proceeds the distance between the shovel and the dump continually increases. The result is that a large portion of the hauling equipment is idling when the cut is begun, but before the end is reached the shovel is waiting a considerable portion of the time. A better method which can frequently be employed is to begin the fill at the balance point or a sufficient distance away from the beginning of the cut, so that as the work proceeds the dump and the shovel will both progress in the same direction and with a haul sufficient to utilize the full equipment to the best advantage. Whenever possible the placing of fills should be so planned that the idle-time loss for the hauling equipment will be reduced to a minimum. Surplus teams may often be used with fresnoes to do cleaning-up work usually done with the shovel.

Various types of hauling equipment are used with the power shovel, the most common of which are teams and bottom-dump wagons, motor trucks, and large tractor-drawn bottom-dump wagons. Teams are used more frequently than any other type, and in general, are very satisfactory for short to moderate hauls. The time required for turning, dumping, and maneuvering is relatively short and their average round-trip speed fairly constant at from about 220 to 240 feet per minute over a wide range of road conditions. On jobs which have very wide variations in the lengths of haul, and especially if the longer hauls involve considerable quantities, the use of teams is apt to be uneconomical unless the conditions are such that either extra teams can be hired as needed or else that when the shorter hauls are reached the otherwise idle teams can be satisfactorily utilized on wheeler or fresno work.

Some of the difficulties of team hauling, especially for the longer hauls, would no doubt be reduced by a more general use of the 2-cubic-yard three-horse wagons. Where team hauling is used the daily cost of remaining idle may readily reach one half the normal daily operating cost. Consequently, with a team outfit it is generally very important that work be carried on whenever it is at all practicable to do so.

CONCLUSIONS REACHED CONCERNING TRUCK AND TRACTOR OPERATION

Heavy trucks, on the other hand, are apt to prove expensive on very short hauls. Three or five ton trucks are rather generally used in some sections. Where the prevailing hauls are moderate to long and the road, dump, and cut can be suitably maintained, these vehicles give good service. Trucks should be equipped with a quick-acting dumping mechanism which will raise the body to a high angle. Since operating room is restricted on most construction jobs, trucks with a short wheel base are generally found to turn faster both at the dump and the shovel—a very important matter on short-haul work where speed is of but minor importance. On long hauls speed becomes an extremely important element. Yet many jobs have been found where the road conditions were such that the average round-trip

speed of the trucks was as low as 300 or 400 feet per minute.

Except where the trucks were equipped with pneumatic tires no job has yet been found where trucks of this class could consistently maintain an average round-trip speed of over 8 miles an hour, or about 700 feet a minute. Generally the speed has been below 6 miles an hour. Where heavy trucks are used more attention to road conditions would be profitable, as would the use of the dual-type pneumatic tires for the rear wheels. Whenever the turning time on short-haul work is long and the operating speed relatively low, the output of the truck can usually be considerably increased by backing the loaded vehicle to the dump and returning it forward to the shovel. Cases have been found where this method proved advantageous up to a haul of over 800 feet. On very short hauls the output of the truck can sometimes be almost doubled in this way.

Bottom-dump wagons of 5 and 6 cubic yards capacity drawn by crawler tractors have been found to be very



LOADING A TRAIN OF TRACTOR-DRAWN DUMP WAGONS

efficient under a wide variety of conditions. Usually two of these wagons can be drawn by one good 10-ton crawler-type tractor. A good tractor operator can handle one of these trains effectively under conditions encountered in ordinary highway grading work. While the operating speed is rather low—about 275 to 325 feet per minute—the dumping and turning time is low, so that two of these trains can ordinarily handle the full output of a $\frac{3}{4}$ -yard shovel up to a haul of from 600 to 800 feet in good common excavation, and to a correspondingly greater distance in material which is more difficult to dig. Each additional train will extend the hauling distance by from 800 to 1,000 feet.

Where crawler tractors are used to draw large dump wagons, only skilled operators who will take an interest in the work should be employed. Since the number of tractor trains required for the economical operation of the ordinary highway grading job rarely exceeds three, it is very important that they be maintained in proper condition and operated with great regularity.

Power-shovel operation in highway grading work involves a large number of repetitive operations. The dipper must be loaded, swung, and spotted over the hauling unit, the load dumped, and the dipper then returned for another load. From time to time the shovel must be moved forward or maneuvered so as to keep it within easy digging reach of the material. The hauling units must be brought into position and loaded, the load hauled to the dump—at the dump turning and backing is often necessary. The load must then be dumped and the vehicle returned to the cut where turning and maneuvering is frequently necessary

to get into position for receiving another load. As these operations are repeated over and over again throughout the day, it is clear that if a few seconds, or even a fraction of a second, are regularly lost on any one operation, the total loss during the course of the day will be large. If an operator working regularly on a 20-second cycle slows up only just enough to regularly add one second to each of the major operations of loading, swinging, dumping, and returning the dipper, the output will be cut from three to two and one half dippers per minute. Or, if the regular unhampered output is 90 wagonloads per hour, and the drivers of the hauling

units slow down so as to delay the shovel only five seconds each time in getting into place to be loaded, then the output will be cut to 80 loads per hour.

Definite stops are, of course obvious and every contractor makes more or less determined efforts to eliminate or reduce them. But a power-shovel outfit may operate all day without a single definite stop and yet not produce more than half the yardage it is capable of producing simply because the management is not aware of the effect on production of the constant loss of seconds or even fractions of seconds in the numerous repetitive operations.

FACTORS AFFECTING THE OPERATING CYCLE

The operating cycle of the power shovel consists of the consecutive actions of (1) loading the dipper, (2) swinging it over the wagon or other hauling unit, (3) dumping it, and (4) swinging it back to loading position. From time to time the shovel must also be moved forward so as to keep within easy digging reach of the face of the cut. This, however, is not a part of the regular operation cycle, but rather a necessary interruption, the relative frequency of which varies principally with the depth of the cut.

Efficiency in power-shovel operation is dependent on the operator and on the shovel itself. A first-class operator may be able to secure fair production with an old or a second-rate shovel, but a poor operator is a heavy handicap even with the very best equipment. It is hoped the records of performance which this series of papers contains may help constructors to increase their present rates of shovel production and the power-shovel manufacturers to so perfect their shovels as to meet still better those conditions which are prerequisite to high rates of production.

It has been stated that under ordinary field conditions the fastest obtainable operating cycle in good common excavation is in the neighborhood of 15 seconds when the swing is 90°. The importance of approaching this limit as closely as possible can hardly be overstated. A 15-second cycle consistently maintained will yield the large output of 240 dipper loads an hour. To attain a 15-second cycle it is necessary to load the dipper regularly in 4 to 5 seconds, to swing it over the wagon in 4 to 5 seconds, to dump it in 1 to 1½ seconds, and return it again to loading position in 4 to 5 seconds. Lengthening the cycle time to 20 seconds drops the output to 180 dipper loads an hour—a reduction of 25 per cent. If the cycle is lengthened to 25 seconds, the best the shovel can turn out is 144 dipper loads an hour, while if a 30-second cycle obtains the output can not exceed 120 dipper loads.

The difference between operation on, let us say, a 15-second cycle and on a 20-second cycle is often a matter of a second or so in loading, a slight hesitation during the swing, with perhaps a bit of delay in spotting over the wagon—delays which may not be noticed except with the aid of extended stop-watch readings. It is not surprising to find that slow operators are sometimes rated as fast because the contractor has nothing definite with which to compare their work and to find that fast operators are sometimes being discredited because job conditions or methods of job management over which they have no control hold down the output.

Examples of the full operating-cycle time where loading was at the side of the shovel are shown in Figures 1 to 5. A detailed analysis of each of the four parts of the operating cycle seems advisable in order to establish more clearly the parts played, respectively, by the operator, the machine, and the material in determining and controlling the rate of production. This will be followed by a somewhat similar analysis of the additional factors affecting production which are more or less completely under the control of the management.

The first activity of the operating cycle is loading the dipper and its effectiveness is dependent on the size of the load and the time taken in securing it. To sacrifice, say, 10 per cent of the size of the dipper load in order to increase the number of dippers by 10 per cent results in a loss in material dug and smaller loads for the hauling units. The smaller the time of loading the dipper as compared with the time of the entire cycle, the greater is the importance of securing a full dipper. It is more important to try for a full dipper when the swing is long than when it is short. In very shallow cuts a low boom will aid in securing a fuller dipper, as will a high boom in a deep cut.

In making these studies determinations have been made as to the number of dipper loads and quantity of material moved under various conditions. Quantities were determined from careful cross-sectioning and are thought to be large enough to represent average conditions. Table 1 gives the results obtained on several jobs. For a so-called ¾-yard bucket having a capacity of approximately three-fourths cubic yard when struck horizontally in line with the top of the teeth and the top of rear edge, the average dipper load may vary from 0.3 to 0.8 cubic yard, depending on the material and the skill of the operator. In fair to good common reasonably free from roots and boulders, a good operator working under favorable conditions should dig an average of 0.5 or 0.6 cubic yard per dipper load. In poorly blasted rock or shale, very rooty and stumpy soils, and certain tough, moist clays, the average load may be only 0.35 cubic yard or even less in exceptional cases. In shallow cuts the average load is likely to be low. Materials which bulk considerably when broken up or which lack cohesion, and will not heap up on the dipper, are apt to show a low average yield per dipper load.

In general, the largest average dipper load can be secured from cuts of moderate depth, from 5 to 7 feet being perhaps the most advantageous in materials not needing to be blasted. The size of the dipper load is sometimes affected by the position in which the hauling units are loaded. In loose, friable materials which spill

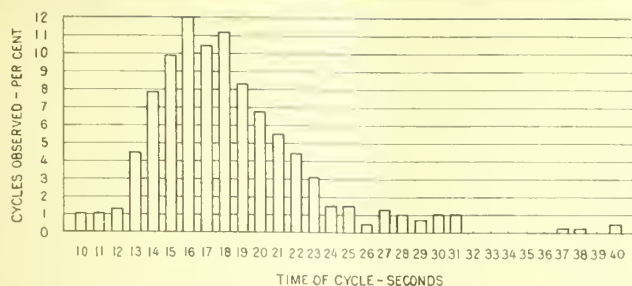


FIG. 1.—DIAGRAM SHOWING PERCENTAGE OF SHOVEL CYCLES PERFORMED IN VARIOUS TIME INTERVALS. BASED ON 383 COMPLETE CYCLES (4 GREATER THAN 40 SECONDS) OF A $\frac{3}{4}$ -YARD SHOVEL WORKING IN AN $8\frac{1}{2}$ -FOOT CUT OF BLASTED SHALE AND LOADING TRUCKS AT SIDE. AVERAGE TIME PER CYCLE, 18.9 SECONDS

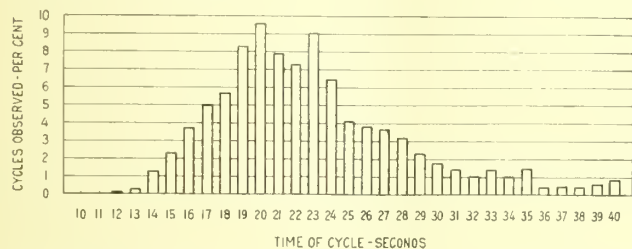


FIG. 2.—DIAGRAM SHOWING PERCENTAGE OF SHOVEL CYCLES PERFORMED IN VARIOUS TIME INTERVALS. BASED ON 1,058 COMPLETE CYCLES (43 WERE OVER 40 SECONDS AND NOT SHOWN) OF A $\frac{3}{4}$ -YARD SHOVEL WORKING IN 1 TO 5 FEET OF A STICKY CLAY AND WITH AN ANGLE OF SWING OF FROM 45° TO 90° . AVERAGE TIME PER CYCLE, 23.9 SECONDS

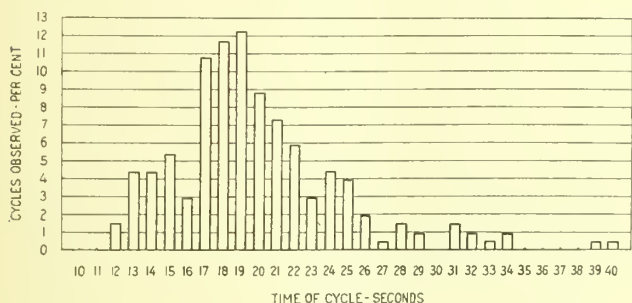


FIG. 3.—DIAGRAM SHOWING PERCENTAGE OF SHOVEL CYCLES PERFORMED IN VARIOUS TIME INTERVALS. BASED ON 204 COMPLETE CYCLES (5 WERE OVER 40 SECONDS AND NOT SHOWN) OF A $\frac{3}{4}$ -YARD SHOVEL WORKING IN FROM 8 INCHES TO 2 FEET OF LOAMY CLAY WITH AN ANGLE OF SWING OF FROM 45° TO 90° . AVERAGE TIME PER CYCLE, 20 SECONDS

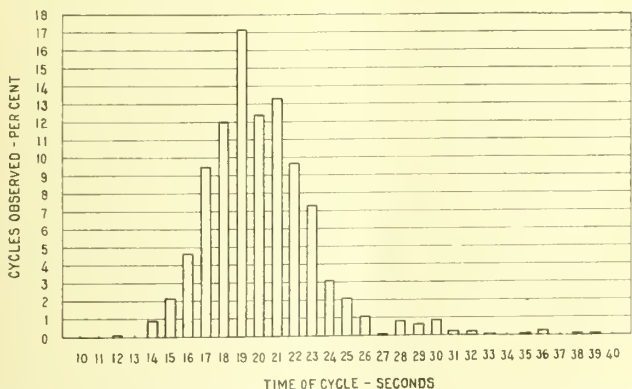


FIG. 4.—DIAGRAM SHOWING PERCENTAGE OF SHOVEL CYCLES PERFORMED IN VARIOUS TIME INTERVALS. BASED ON 734 COMPLETE CYCLES (3 WERE OVER 40 SECONDS AND NOT SHOWN) OF A $1\frac{1}{8}$ -YARD SHOVEL WORKING IN 2 TO 6 FEET OF CLAY WITH A FEW BOULDERS. LENGTH OF SWING, 45° TO 90° . AVERAGE TIME PER CYCLE 20.3 SECONDS

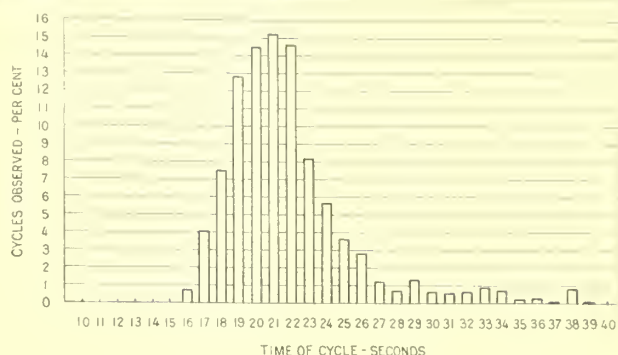


FIG. 5.—DIAGRAM SHOWING PERCENTAGE OF SHOVEL CYCLES PERFORMED IN VARIOUS TIME INTERVALS. BASED ON 1,322 COMPLETE CYCLES (20 WERE OVER 40 SECONDS AND NOT SHOWN) OF A $1\frac{1}{8}$ -YARD SHOVEL WORKING IN FROM 1 TO 6 FEET OF CLAY WITH A FEW BOULDERS WITH A SWING OF FROM 45° TO 90° . AVERAGE TIME PER CYCLE, 22 SECONDS

TABLE 1.—Number of dipper loads and quantity of material moved under various conditions

Type of shovel	Capacity	Character of material	Quantity moved	Dipper loads	Average loading
Steam	Cu. yds. $\frac{3}{4}$	Light moist clay, free from roots and stones.	57	147	0.39
Do.	$\frac{3}{4}$	do.	114	223	.51
Do.	$\frac{3}{4}$	do.	85	170	.50
Do.	$\frac{3}{4}$	Light moist clay, with some shale.	65	148	.44
Do.	$\frac{3}{4}$	Loamy clay, with 25 per cent loose rock.	19	50	.38
Do.	$\frac{3}{4}$	do.	63	156	.40
Do.	$\frac{3}{4}$	Sand-clay.	49	82	.60
Do.	$\frac{3}{4}$	do.	93	150	.62
Do.	$\frac{3}{4}$	Loamy to hard clay.	49	85	.58
Do.	$\frac{3}{4}$	Loamy to sandy clay.	50	141	.35
Do.	$\frac{3}{4}$	Loamy to clay.	60	157	.38
Do.	$\frac{3}{4}$	do.	39	72	.53
Do.	$\frac{3}{4}$	Gneiss-granite, poorly blasted.	985		.33
Do.	$\frac{3}{4}$	Wet sticky clay, with a few stumps.	1,167	1,745	.67
Do.	$\frac{3}{4}$	Moist to wet sand-clay.	1,468	1,825	.80
Do.	$\frac{3}{4}$	Sandstone, well blasted.	219	632	.35
Do.	$\frac{3}{4}$	do.	1,120	2,599	.43
Do.	$\frac{3}{4}$	Moist clay, with a few small surface boulders.	518	794	.65
Do.	$\frac{3}{4}$	Very wet clay.	588	990	.59
Do.	$\frac{3}{4}$	Wet clay, with small stumps.	100	210	.48
Do.	$\frac{3}{4}$	Sandy gravel, with some hard chunks of shale.	1,683	4,099	.41
Do.	$\frac{3}{4}$	Dry loamy clay.	162	309	.53
Do.	$\frac{3}{4}$	do.	29	71	.41
Do.	$\frac{3}{4}$	Granite-gneiss, poorly blasted.	1,335		.40
Gas.	$\frac{3}{4}$	Loamy clay, moist, with a few roots.	356	583	.61
Do.	$1\frac{1}{8}$	Sandstone, blasted.	1,810	3,448	.53
Do.	$1\frac{1}{8}$	Dry clay, with a few boulders.	1,523	2,892	.53
Do.	$1\frac{1}{8}$	Dry clay, with surface boulders.	635	996	.64
Do.	$1\frac{1}{8}$	70 per cent large boulders and 30 per cent dry clay.	381	667	.57
Do.	$1\frac{1}{8}$	10 per cent dry clay, 20 per cent loose rock, 70 per cent solid rock, blasted.	2,759	4,384	.63
Do.	$1\frac{1}{8}$	Wet sticky clay, with a few surface boulders.	1,364	2,396	.57
Do.	$1\frac{1}{8}$	20 per cent dry clay, with 80 per cent sandstone, well blasted.	474	784	.60
Steam.	$\frac{3}{4}$	Sandy clay and clay loam, with some stone.	1,555	3,504	.44
Gas.	$\frac{3}{4}$	80 per cent sandstone, poorly blasted, with 20 per cent clay.	363	788	.46
Do.	$\frac{3}{4}$	do.	3,010	6,646	.46

readily, a long swing and especially loading near the limiting height to which the shovel can reach is certain to reduce the amount of material moved per dipper load. Since the rate of operation in materials of this kind should be around 200 dippers an hour, even small differences in the size of the dipper load become important.

Three cases were studied where the contractors in a laudable effort to secure a large yardage per dipper load had replaced regular $\frac{3}{4}$ -yard dippers with $1\frac{1}{4}$ -yard dippers. This proved a decided handicap, except possibly in extremely soft and easy digging, as the power was insufficient to force the large dipper into the material. Not only was the production less than the normal for a regular $\frac{3}{4}$ -cubic yard shovel, but time

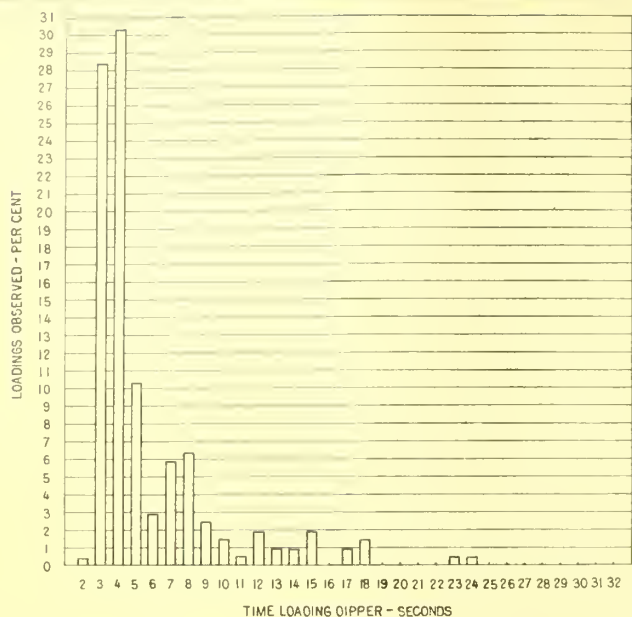


FIG. 6.—DIAGRAM SHOWING PERCENTAGE OF LOADING OPERATIONS PERFORMED IN VARIOUS TIME INTERVALS. BASED ON 204 LOADINGS OF A $\frac{3}{4}$ -YARD DIPPER (3 WERE OVER 32 SECONDS) IN A $1\frac{1}{2}$ TO 2 FOOT CUT OF LOAMY CLAY. AVERAGE TIME, 6.1 SECONDS

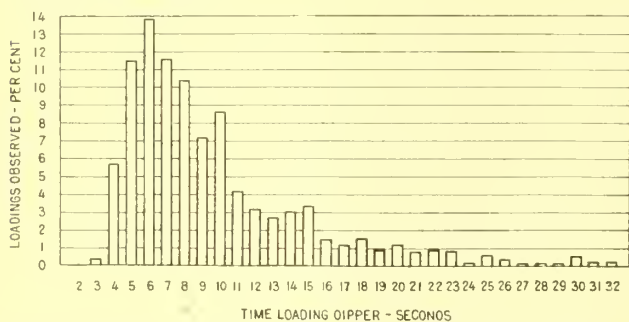


FIG. 7.—DIAGRAM SHOWING PERCENTAGE OF LOADING OPERATIONS PERFORMED IN VARIOUS TIME INTERVALS. BASED ON 1,058 LOADINGS (18 WERE OVER 32 SECONDS) OF A $\frac{3}{4}$ -YARD DIPPER WORKING IN 1 TO 5 FEET OF STICKY CLAY. AVERAGE TIME, 10.29 SECONDS

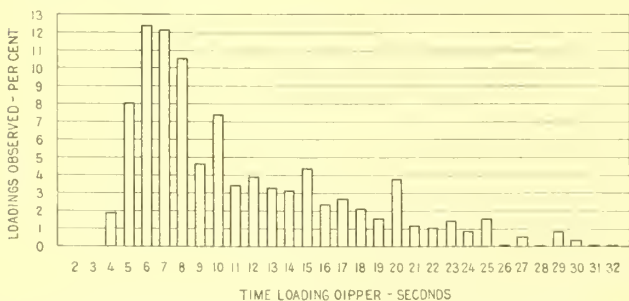


FIG. 8.—DIAGRAM SHOWING PERCENTAGE OF LOADING OPERATIONS PERFORMED IN VARIOUS TIME INTERVALS. BASED ON 658 LOADINGS (16 WERE OVER 32 SECONDS) OF A $\frac{3}{4}$ -YARD SHOVEL WORKING IN 2 TO 6 FEET OF STICKY CLAY. AVERAGE TIME, 12.2 SECONDS

losses due to breakage and repairs were high. This seems to indicate that for general highway work increased production is not to be had by increasing the size of the dipper above that for which the shovel is designed.

The time required to load the dipper often varies considerably from the average, as shown in Figures 6 to 12 and Tables 2 to 4. Each of the graphs shown

covers a number of observations and shows the number of times the dipper was loaded in any given number of seconds. They show that a few of the loading times took very much longer than the others and reduced the average very materially.

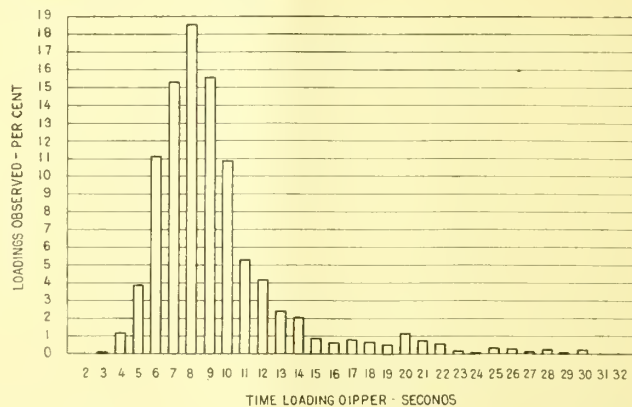


FIG. 9.—DIAGRAM SHOWING PERCENTAGE OF LOADING OPERATIONS PERFORMED IN VARIOUS TIME INTERVALS. BASED ON 1,322 LOADINGS (15 WERE OVER 32 SECONDS) OF A $1\frac{1}{8}$ -YARD DIPPER WORKING IN $1\frac{1}{2}$ TO 5 FEET OF CLAY AND LOAM WITH A FEW BOWLERS. AVERAGE TIME, 9.67 SECONDS

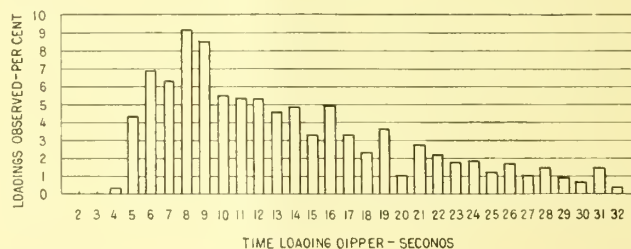


FIG. 10.—DIAGRAM SHOWING PERCENTAGE OF LOADING OPERATIONS PERFORMED IN VARIOUS TIME INTERVALS. BASED ON 763 LOADINGS (21 WERE OVER 32 SECONDS) OF A $\frac{3}{4}$ -YARD DIPPER WORKING IN 1 TO $4\frac{1}{2}$ FEET OF LIGHT LOAM. AVERAGE TIME 16.5 SECONDS

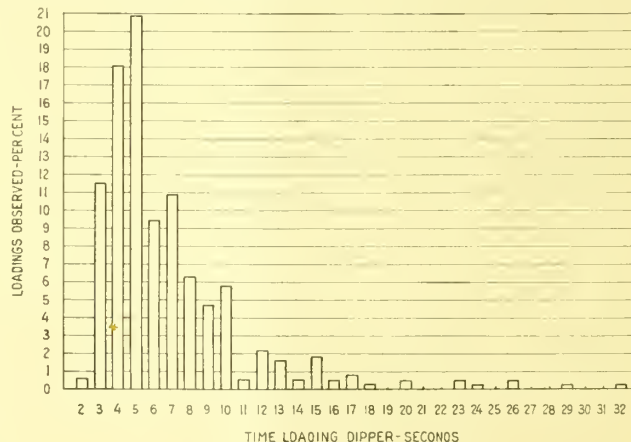


FIG. 11.—DIAGRAM SHOWING PERCENTAGE OF LOADING OPERATIONS PERFORMED IN VARIOUS TIME INTERVALS. BASED ON 383 LOADINGS (6 WERE OVER 32 SECONDS) OF A $\frac{3}{4}$ -YARD DIPPER WORKING IN AN 8 TO 9 FOOT CUT OF BLASTED SHALE. AVERAGE TIME 6.9 SECONDS

Figure 6 shows a job in loamy clay where the performance was excellent. A comparatively large number of dipper loads were secured in four seconds. The average loading time was 6.1 seconds, and there were only a relatively small number of dipper loads which took a long time to secure—an indication that a good, consistent operator was handling the shovel. The

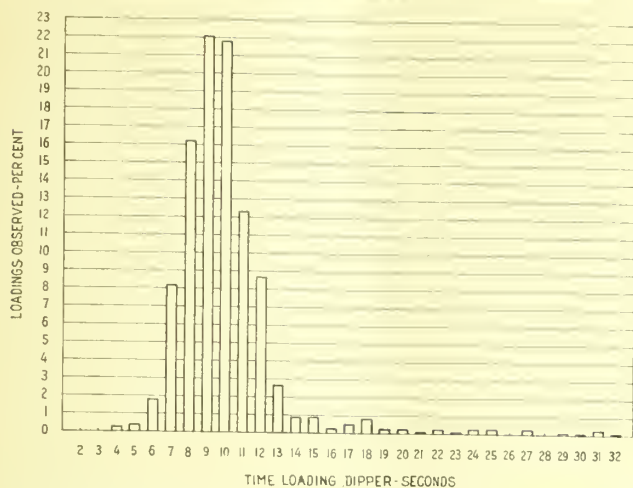


FIG. 12.—DIAGRAM SHOWING PERCENTAGE OF LOADING OPERATIONS PERFORMED IN VARIOUS TIME INTERVALS. BASED ON 734 LOADINGS OF $1\frac{1}{8}$ -YARD SHOVEL WORKING IN $2\frac{1}{2}$ TO 7 FEET OF CLAY WITH A FEW BOWLERS

average time during this study for the full cycle of loading, dumping, and return, however, was 20 seconds, since the swing was rather long and the material sufficiently moist to hang somewhat in the dipper.

JOB CONDITIONS DETERMINE WHETHER MORE THAN ONE PASS OF DIPPER IS JUSTIFIED

Special attention is drawn to Figures 9 and 11 as examples of good, consistent operation. In the case shown in Figure 8, work was in somewhat sticky clay in which it was hard to secure a full load. In about



LIGHT WORK WHERE MORE THAN THE AVERAGE AMOUNT OF TIME IS REQUIRED TO FILL THE DIPPER

50 per cent of the cases observed, one lift of the dipper was used to secure a load, and in these cases the dipper load was secured in an average of less than seven seconds. In the remaining cases, two or three or even four passes were made to fill the dipper to the satisfaction of the shovel runner. The result was an occasional extension of the time required in filling the dipper to as much as a full minute. The average time for all of the studies was 12.2 seconds. Figures 6, 9, and 12 show the results where the operator regularly made only one pass with the shovel, loading whatever he was able to secure. In these figures are noted the small number of very long readings, and such as occurred were due principally to stumps or heavy rocks.

Table 2 shows the time required in making two or more passes to load the dipper where the shovel was

operating in ordinary gravelly clay in moist to wet condition. The average time for loading the dipper when only one pass was necessary was 7.2 seconds. When two passes were required the time rose to 14.1 seconds and to 21.2 seconds for three passes. Thus, each additional pass increased the time required to load the dipper by 95 per cent above the time required where only one pass was necessary.



LOOSE, DRY MATERIAL FALLING FROM TOP OF DIPPER

TABLE 2.—Time required in making multiple passes in loading dipper in a moist to wet gravelly clay

Number of passes		Number of observations	Average time per dipper load
			Seconds
1		1,332	7.2
2		298	14.1
3		82	21.2
4		32	27.6
Total or average.....		1,744	9.4

But production is the product of the number of dipper loads and the average quantity per dipper load. Consequently, the value of making additional passes is dependent on the amount of material which such passes will add to the dipper load and the speed at which the shovel is operating. If a $\frac{3}{4}$ -yard shovel is operating in fairly good common in which the average dipper load is about 0.5 cubic yard of material as measured in place, and the average operating cycle is 20 seconds, then production is at an average rate of 0.025 cubic yard per second. If a second pass to fill an occasional dipper is to be profitable, it must serve to increase the dipper load at least at this rate, for the time required to make the extra pass. Thus, if six seconds are required to make an extra pass under the above conditions, it would not be warranted unless at least 0.15 cubic yard could be added to the load. In other words, whenever the first pass secured as much as three-fourths of an average dipper load a second pass would not be warranted, considering the case from the viewpoint of shovel output alone.

NOT POSSIBLE TO FORMULATE DEFINITE RULES OF PROCEDURE

Definite rules can not be formulated since each operation is interrelated with many other possible conditions surrounding the entire job. It is possible to show the principles which apply and by means of which the proper procedure can be determined. It has been stated that the importance of securing a full

dipper load is greatest when the time required to load the dipper is in smallest ratio to the total shovel cycle, and vice versa. Therefore, more attention to securing a large dipper load is justified when loading at the rear of the shovel than when the loading is at the side. In shallow cuts, where much skimming is required, dipper loads are almost certain to average a low quantity. Keeping the boom lower than normal will generally help in securing larger loads. On such work a considerable amount of time is consumed in moving the shovel. If there is any considerable amount of it, the contractor may well consider the advisability of using some other method on such portion of the work.

It is not possible to state categorically that any one system of operation is always to be preferred. In the case illustrated by Figure 8 the swing was long and there was not enough hauling equipment on the job; so lengthening the dipper cycle in order to secure a full dipper load did not, in fact, subtract much from the shovel output, while it tended to insure a full wagon-load every trip which increased the job output. In the cases shown in Figures 9 and 12 the shovel was a 1½-yard machine and two full dippers overloaded the wagons. As long as the operator could get one full dipper load out of two, he could send the wagons to the dump well loaded without making multiple passes. On this job the wagon supply, generally speaking, was above average so that fast operation was desirable.

But, aside from the conditions prevailing on these two jobs and the rates attained, these two figures show several of the general characteristics of what, under normal working conditions, would be good operation and poor operation. A good operator working in anything like good material under favorable conditions of depth and face of cut obtains a loading diagram for his dipper like those in Figures 6, 9, and 12. A poor operator always has an operating record more or less like those in Figures 8 and 10. On the jobs represented by the first set of graphs the shovel was so handled that the desired load was obtained quickly with one pass of the shovel. On the jobs represented by the second set the rate was slower, and frequently two, three, or more passes were made to fill the shovel.

SLOW LOADING MAY BE CAUSED BY SEVERAL FACTORS

The position of the shovel with reference to the face of cut is in no small degree responsible for the repeated passes some shovel runners make in filling the dipper. The dipper is actuated by two separate and distinct motions—one, known as the “hoist,” tends to raise the dipper in a vertical circle about the point of intersection of the boom and the dipper stick, while the other, known as the “crowd,” controls the length of the radius of the arc in which the “hoist” moves the dipper. In loading, the “crowd” is used to force the dipper against the face of the cut, and on the swing to spot the dipper correctly over the hauling unit. When the dipper stick is in a vertical position the combined motion of the “crowd” and the “hoist” can drive the cutting edge of the dipper almost straight forward several feet, and when the dipper stick is horizontal the crowd serves to hold the cutting edge of the dipper hard into the bank.

When loading the dipper from a bank less than 2 feet high, direct thrust forward into the bank is required, while on a bank 6 or more feet high the loading is generally best done by a longer swinging motion in which a comparatively thin slice is cut from the bank. For some reason it appears to be difficult to find opera-

tors who will perform both operations equally well. To make the shovel function smoothly where the bank is low, it must stand close to the bank with the boom somewhat lower than normal and must be moved forward frequently. This is because the forward thrust of the dipper resulting from the proper combination of the “hoist” and “crowd” only reaches a relatively short distance. Cutting from a high bank is best done after the dipper has begun to turn definitely upward in its swing, which requires that the shovel should stand well back from the bank with the boom high. The superintendent who will watch this matter closely and drill his shovel runners in the proper positioning of the shovel for effective dipper loading will find the results gratifying.

In addition to these causes of slow loading, viz, poor handling of the dipper itself and improper position of the shovel with reference to the face of the cut, the material itself is responsible for a good deal of slow digging. Banks composed of good-sized rocks embedded in a stiff clay are particularly troublesome. The runner can not see such rocks, and when the dipper strikes one it may be necessary to draw back and try again. Often two or three passes must be made—sometimes twice that many—before either a load of loose material is secured or the exact position of the rock defined so that it can be picked up.

Tables 3 and 4 show something of the effect of the material on the time required for filling the dipper. Table 3 contains a few good illustrations of fast work in good common by shovel No. 1. A comparison of these tables will give a very fair impression of the difference between work in good ground and work in difficult material. They do not show the quantities moved, but, in general, fast operation in good material was accompanied by large dipper loads and the quantity decreased with difficulty in loading as indicated by the time factor.

TABLE 3.—Effect of material on time required to load a ¾-yard dipper as indicated by one-hour stop-watch studies with same operator throughout on each shovel

SHOVEL NO. 1		
Kind and character of material	Time to load dipper	Height face
	Seconds	Feet
Light sandy loam, free from roots and stones	3.3	10.0
Do	3.3	7.0
Loamy clay, free from roots and stones	4.5	6.5
Do	4.7	6.0
Do	4.8	10.0
Light clay and loam top soil, no roots or stones	4.9	2.0
Light clay with small amount of soft shale	5.0	5.0
Loam	5.3	5.0
Light clay, free from roots and stones	5.6	10.0
Light clay with small amount of shale	5.7	10.0
Do	5.8	4.0
Light clay with increasing amount of shale	5.9	7.0
Loamy clay, with some roots	6.0	6.5
Ordinary clay, free from roots and stones	6.2	4.5
Loamy clay	6.5	0.3-1.5
Light clay and soft shale	6.6	8.0
Loamy clay with loose rock in old roadbed	8.2	2.0
Clay with 50 per cent loose shale	10.7	6.5
Hard clay with loose rock in old roadbed	15.9	0.25-2.5
Hard clay with loose rock	16.3	2.0

SHOVEL NO. 2		
Light, loamy clay, no stones or roots	5.2	5.5
Do	5.3	4.0
Light clay, free from stones or roots	5.7	4.0
Light clay, practically free from roots and stones	6.0	3.5
Light clay with old hard roadbed on one side	6.7	6.0
Do	6.9	4.0
Light to medium clay	7.5	4.5
Clay and soft shale	8.2	2.0
Light clay with small amount of rock, side hill cut	9.2	0-4.0
Clay and soft shale	9.4	7.0
Sandstone, soft enough to crumble in hand	11.0	11.0

TABLE 4.—Time required to load dipper in various kinds of material. Each entry shows the number of one-hour studies in which the average time of loading fell within the range indicated. Studies were made on a great many typical jobs

Average time to load dipper (seconds)	Loam, loamy, sandy, gravelly, or friable materials, practically free from roots, boulders, etc.	Ordinary clays, ordinary soils, and friable materials with few roots or loose rock, etc.	Fairly hard or tough clays, ordinary clay with some loose rock, shale, etc.	Mixtures of clay and loose rock, soft shale, and hard and tough clays, etc.	Well-blasted rock, shale, etc.	Fairly well blasted rock, loose rock and boulders, hard or tough clay with rocks, etc.	Clay-gravel in 6 to 9 foot cut with 2 feet of hard frost on surface.	Poorly blasted rock or shale, boulders, hardpan, etc.
3 to 4	1							
4 to 5	7							
5 to 6	18							
6 to 7	2							
7 to 8		18						
8 to 9		12						
9 to 10		5						
10 to 11			11					
11 to 12			13					
12 to 13			26					
13 to 14				6	6			
14 to 15				21	4			
15 to 16				11	9		3	
16 to 17				7	5		2	
17 to 18					3	4	1	
18 to 19					2	7	1	
19 to 20						10		1
20 to 22						7	4	
22 to 24						6		3
24 to 26						1	2	9
26 to 28							1	3
28 to 30								7
30 to 35								4
35 to 40								1
Average time of loading in seconds	5.2	7.1	9.7	11.9	12.4	17.6	17.4	23.9

LARGE ANGLE OF SWING CAUSES CONSIDERABLE TIME LOSSES

Figures 13 to 15 show the time consumed in making the swing and returning the dipper for various angles of swing on three jobs. Under perfect operating conditions curves showing the rate of swing would probably be straight lines except for the influence of acceleration. That the points for one set of final observations do not all fall on the same line is due to the fallibility of the operator and the many other factors influencing performance; each individual swing ordinarily being subjected to a set of conditions different in some particular from the others.

The curves have not been projected to intercept the x -axis, since measurements were not made of the time loss in accelerating the dipper, but inspection of these and other graphs not shown indicates that the x -intercept or lag in getting the dipper started on the swing is variable for different shovels. It appears that the types of shovels with a slow-swing speed have a shorter lag, so that the actual time required for the swing and return, as shown in Table 5, is often as short or shorter for the slow-speed types so long as the angle of swing is small, but on the longer swings the higher speed types are considerably faster. Figures 16, 17, and 18 also show the variations found in the swing and return time under actual field operation.

The portion of the swing and return time which has here, for want of a better term, been designated as the "lag" is evidently the sum of two or more factors. Time is required for the operator to react and manipulate the necessary mechanism, and it is also required to accelerate the shovel and its load

TABLE 5.—Comparison of combined swing and return time for various types of power shovels

[The values are averages from field studies under ordinary operating conditions]

Angle of swing	Gas shovel	Steam shovel	Gas and air shovel	Gas shovel
	Seconds	Seconds	Seconds	Seconds
30°	8.2	8.0	8.2	5.7
60°	10.4	9.0	9.9	8.4
90°	12.5	10.0	11.5	11.2
120°	14.6	11.0	13.2	13.9
180°	19.0	12.9	16.5	19.3
240°	23.4	14.9	19.8	24.8
270°	25.5	15.9	21.5	27.5



HANDLING DIFFICULT MATERIAL GREATLY INCREASES THE CYCLE TIME

from rest and again to decelerate it at the end of the swing. While it is very difficult to separate the lag into its purely personal and mechanical factors, the studies show very conclusively that the personal element involved is sufficiently large to warrant careful consideration because of its effect on the rate of production. For a good operator these reaction lags are small—generally less than one second. For a slow operator they will run twice this or even more. The difference of one or two seconds seems like a trifling matter, but the use of an operator who, because of these losses, takes 18 seconds where only 15 are necessary, reduces the rate of production almost 17 per cent. To change an operator making a load every 18 seconds for a man who takes 20 seconds reduces the rate of output about 10 per cent. With a good wagon supply and production running at a high rate, this can easily reduce the value of the output secured from \$20 to \$25 a day—about twice the ordinary wages of a first-class operator. It never pays to hire cheap, poorly trained operators on any sort of work requiring fast, uniform, consistent operation, and there is no point in highway work where this is more true than on power shovels.

Reverting to the swing itself and starting with the 90° swing (loading at the side of the shovel) which in good common and under favorable conditions can be done in 15 seconds, an extension of the swing to 180° (loading back of the shovel) extends the cycle by from three to eight seconds, depending on the type of shovel used and the skill of the operator. As a general average it may be said that loading behind the shovel extends a 15-second cycle to 20 seconds, and by so doing reduces the attainable rate of output 25 per cent.

There is much work where loading at the side of the shovel is impossible, largely because of narrow cuts. Deep cuts sometimes make loading at the side impossible if the whole cut is taken out at one operation. In

spotting the dipper exactly before dropping the load. Hauling units are usually placed for a swing from one side. This is important if rock is being handled, for while a good operator seldom drops any material it

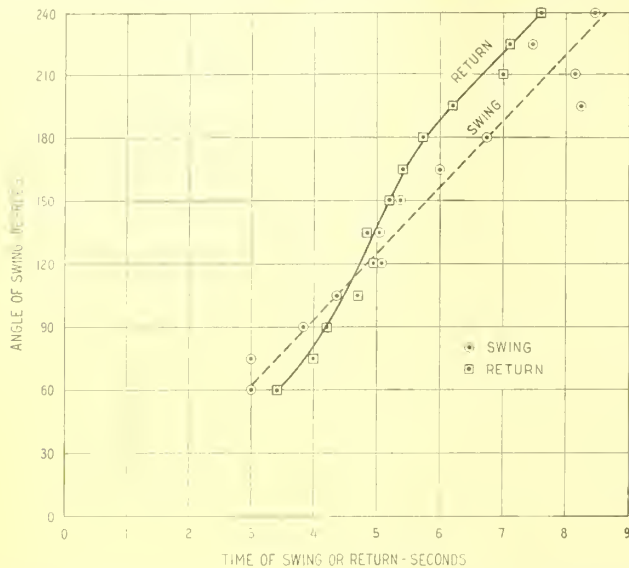
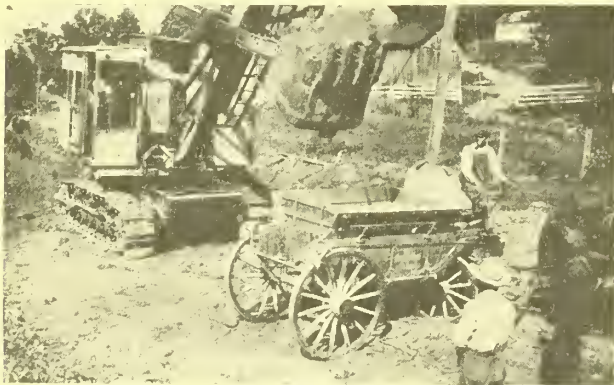


FIG. 13.—INFLUENCE OF ANGLE OF SWING ON TIME OF SWING AND RETURN BASED ON 506 OPERATIONS OF A $\frac{3}{4}$ -YARD SHOVEL HANDLING POORLY BLASTED ROCK. NOTE THAT THE POINTS LOCATED FOR TIME OF SWING ARE MUCH MORE IRREGULAR THAN THOSE FOR TIME OF RETURN. THIS IS EXPLAINED IN PART BY THE EXTRA CARE REQUIRED IN HANDLING BOWLDERS. AVERAGE RATE OF SWING 32° PER SECOND. AVERAGE RATE OF RETURN 46° PER SECOND

general, however, there are many more situations where side loading could be practiced than are now utilized. Many contractors apparently feel that there are so many places where side loading can not be done that



A FULL DIPPER LOAD BEING SPOTTED FOR DUMPING

nothing substantial can be gained by training their men to utilize such opportunities as occur. The advantage of such methods depends largely on the available supply of transportation which will be discussed in a following article.

Extending the average swing to 270° is unqualifiedly objectionable and especially so where slow-swing shovels are used. This extends the cycle to 25 or more seconds and correspondingly reduces the rate of output. Such operation may be caused by a cab arrangement where it is hard for the operator to see out of one side of it. To avoid swinging his load over objects he can not see clearly and running the risk of dropping material on men or animals he can not see, the longer swing is sometimes used. Poor vision also interferes with

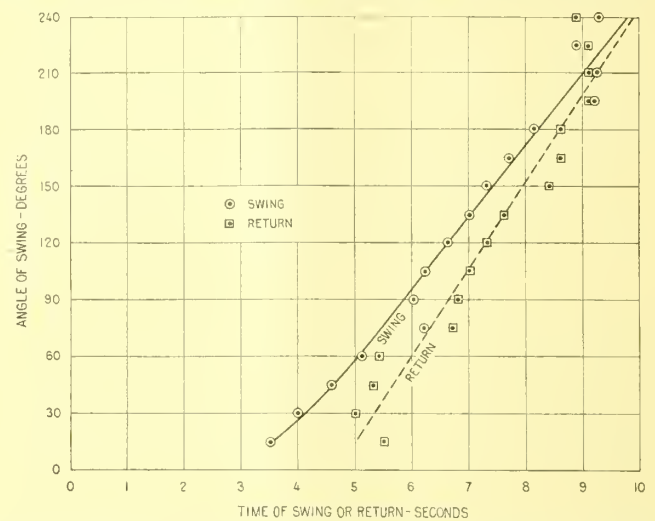


FIG. 14.—INFLUENCE OF ANGLE OF SWING ON TIME OF SWING AND RETURN BASED ON 2,069 OPERATIONS OF A $\frac{3}{4}$ -YARD SHOVEL WORKING IN CLAYEY GRAVEL. AVERAGE RATE OF SWING 37.5° PER SECOND. AVERAGE RATE OF RETURN 46° PER SECOND

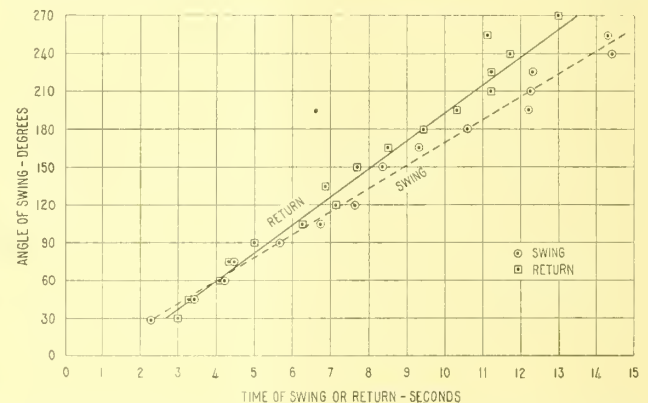


FIG. 15.—INFLUENCE OF ANGLE OF SWING ON TIME OF SWING AND RETURN, BASED ON 1,788 OPERATIONS OF A $\frac{3}{4}$ -YARD SHOVEL WORKING IN GRAVEL AND LOOSE AND BLASTED SHALE. AVERAGE RATE OF SWING, 18° PER SECOND. AVERAGE RATE OF RETURN, 22° PER SECOND

takes very little to seriously injure a man or an animal. As a general rule, it may be said that an average swing of much over 180° is the result of improper equipment or faulty operating methods, but that occasions may arise when it is better to make a few 270° swings than to change the loading position of the hauling equipment.

Table 6 shows the average time used in loading, swinging, dumping, and returning the dipper for several jobs where the swing ranged from 30° to 270° and when the swing was 90° or less. It is apparent that when the point of loading and the point of dumping are within the operator's vision at the same time he can keep his mind far enough ahead of his work so that his general reactions are faster and all of his operations are conducted with more confidence. As he digs his load he determines where he will dump and the manipulations necessary in the process. As he drops it he determines where he will get the next bite, and so on. The saving in time is small per load, but it is enough to make considerable difference in the day's run.

TABLE 6.—Time required to load, swing, dump, and return shovel on various jobs where the swing varied from 30° to 270° and on the same jobs where it did not exceed 90°

ANGLES OF SWING FROM 30° TO 270°						
Shovel	Readings	Load	Swing	Dump	Return	Total cycle
Type A, Delaware County, N. Y.	2,033	12.8	4.6	2.7	4.2	24.3
Do.	1,788	12.7	8.3	4.0	7.5	32.5
Type B, Pike County, Pa.	2,642	9.7	6.1	3.3	6.3	25.4
Type C, Pike County, Pa.	2,069	10.0	7.1	4.4	7.6	29.1
Type D, Columbia County, N. Y.	868	10.3	5.3	3.5	5.4	24.5
Type E, Hughes County, Okla.	1,734	9.4	4.5	2.1	5.5	21.5

ANGLES OF SWING 90° OR LESS						
Type A, Delaware County, N. Y.	1,781	11.9	4.2	2.3	3.8	22.2
Do.	645	13.6	4.2	3.0	4.1	24.9
Type B, Pike County, Pa.	1,625	8.9	4.6	2.9	5.2	21.6
Type C, Pike County, Pa.	634	10.0	5.3	3.5	5.9	24.7
Type D, Columbia County, N. Y.	548	10.0	3.9	3.2	3.8	20.9
Type E, Hughes County, Okla.	1,677	9.3	4.4	2.1	5.4	21.2

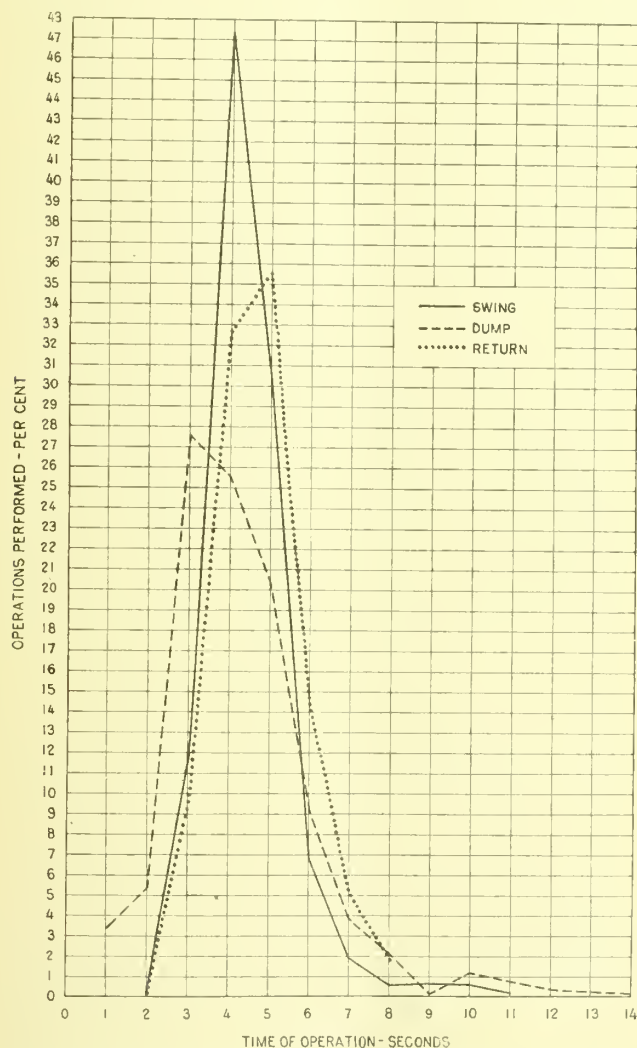


FIG. 16.—PERCENTAGE OF OPERATIONS OF SWINGING, DUMPING, AND RETURNING PERFORMED IN VARIOUS TIME INTERVALS.¹ BASED ON 1,058 CYCLES OF A $\frac{3}{4}$ -YARD SHOVEL WORKING IN STICKY CLAY WITH AN ANGLE OF SWING OF FROM 45° TO 90°. AVERAGE TIME OF SWING 4.42 SECONDS. AVERAGE TIME OF DUMPING, 4.31 SECONDS. AVERAGE TIME OF RETURN, 4.86 SECONDS

¹ Each of these curves (figs. 16 to 19) is in reality formed from the sum of a number of superimposed curves having the general form of the skewed probability curve, but in the interest of clearness the points from actual field readings have been connected by straight lines.

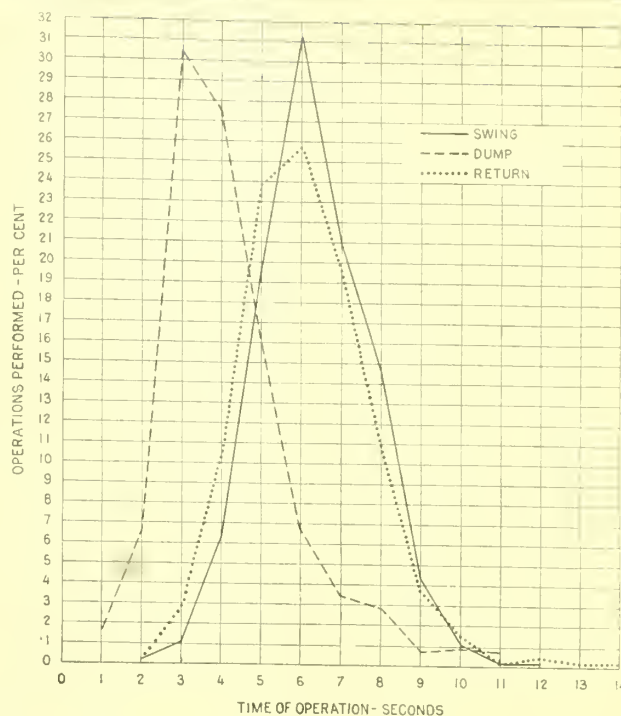


FIG. 17.—PERCENTAGE OF OPERATIONS OF SWINGING, DUMPING, AND RETURNING PERFORMED IN VARIOUS TIME INTERVALS.¹ BASED ON 658 CYCLES OF A $\frac{3}{4}$ -YARD SHOVEL WORKING IN STICKY CLAY WITH AN ANGLE OF SWING OF FROM 150° TO 180°. AVERAGE TIME OF SWING, 6.23 SECONDS. AVERAGE TIME OF DUMPING 4.33 SECONDS. AVERAGE TIME OF RETURN 6.10 SECONDS

DUMPING THE DIPPER

The next item in the list going to make up the full cycle is discharging the load. This is an operation which requires great skill if it is to be done rapidly. The load must not be dropped from too great a height or the wagon will be damaged, and it must not be dropped too soon or too late or much of it will fall outside of the wagon. If the load is composed of loose common—light loam, sand, loamy clay, or rather fine gravel—an experienced shovel runner will drop it just as the swing ends and be ready to start the return as soon as the dipper comes to a stop. In such material the fast operator really takes no time to drop the load, the time consumed being only that needed to stop the dipper and then start it on the return swing, and this can be done regularly in one second. Wet, sticky clays and other adhesive materials often require considerable shaking or jarring to get them out of the dipper. The time required depends on the skill of the operator but more largely on the average amount of shaking and jarring necessary to get them out of the dipper. It is not unusual to find dumping time in sticky materials regularly running as high as five or six seconds, though it necessarily varies with the degree of adhesiveness of the material. Considerable care must be taken by the operator to prevent injury to the wagons or trucks, which naturally slows down the rate of operation and large chunks often hang or wedge in the dipper and require manipulation to release them. Roots and stumps are often very troublesome in this respect.

Table 7 shows how the average dumping time is affected by the kind and character of the materials found from one-hour stop-watch studies. Figures 16, 17, and 18 show the time used in dumping material

from the dipper into the wagons or trucks under typical conditions. Figure 19 shows the average dumping time for a number of classes of material as found from the analysis of 10,200 readings secured on 13 different

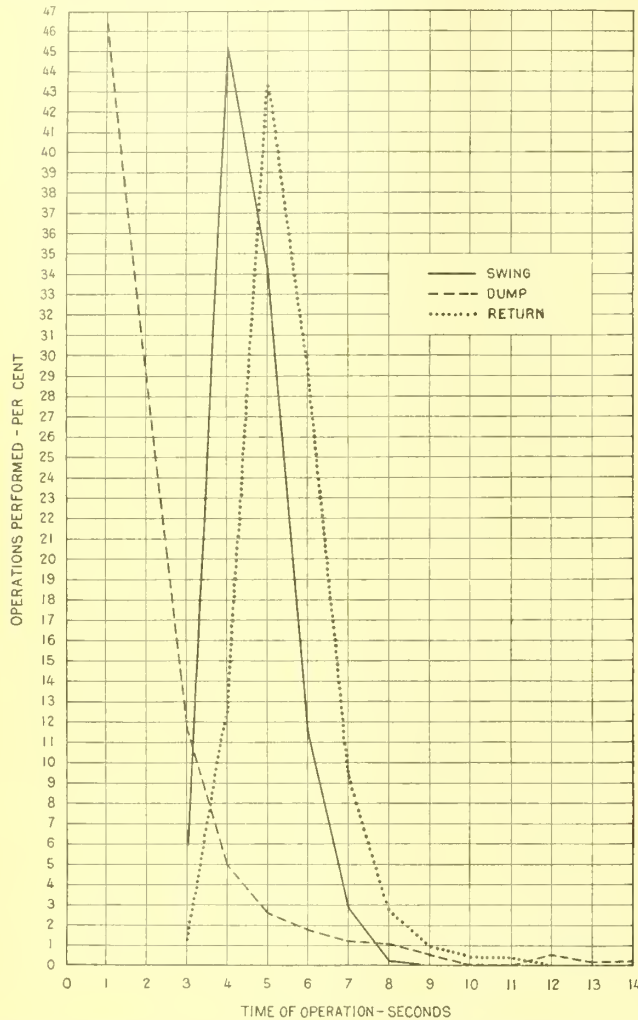


FIG. 18.—PERCENTAGE OF OPERATIONS OF SWINGING, DUMPING, AND RETURNING PERFORMED IN VARIOUS TIME INTERVALS. BASED ON 1,322 CYCLES OF A $1\frac{1}{8}$ -YARD SHOVEL WORKING IN CLAY WITH SOME BOWLDERS WITH AN ANGLE OF SWING VARYING FROM 45° TO 90° . AVERAGE TIME OF SWING, 4.62 SECONDS. AVERAGE TIME OF DUMPING, 2.23 SECONDS. AVERAGE TIME OF RETURN 5.49 SECONDS

jobs with various grades of operators and six different makes of shovels. It will be noted that a considerable percentage of the operations were performed very rapidly.

CRAWLER-TRACTION SHOVELS HAVE BIG ADVANTAGE IN SPEED OF MOVEMENT

The movement of the shovel within the cut to keep within easy reach of the face constitutes a definite limitation to uninterrupted repetition of the dipper cycle and is a check on production which can not be entirely removed. The best that can be done is to train the operator to make these moves as expeditiously as possible. In deep cuts the time required is small, generally less than 1 per cent, where shovels with the latest crawler-type traction are used. In shallow cuts, however, the proportion mounts very rapidly and cases where 8 to 10 per cent of the total time is used in moving the shovel are not uncommon,

TABLE 7.—Effect of character of material on time required to load and dump dipper

Character of material	Observations	Time to dump dipper	Time to load dipper
	Number	Seconds	Seconds
Loam and light clay.....	722	1.0	4.2
Do.....	351	1.5	5.4
Loamy clay and soft shale.....	254	1.9	5.4
Soft shale.....	399	4.2	6.5
Sandy clay.....	249	3.0	7.4
Moist clay.....	96	2.4	8.0
Light clay, wet and gummy.....	173	4.6	8.1
Clay and surface loam.....	692	1.9	8.4
Sandy clay, moist to wet.....	349	4.8	8.8
Well blasted sandstone with 20 per cent light clay.....	229	1.8	9.3
Clay with a few bowlders.....	448	2.1	10.0
Heavy clay, wet and gummy.....	271	5.3	10.4
Clay with some surface bowlders.....	2,892	1.8	10.5
Loam with loose rock and loose shale.....	369	2.8	10.5
Do.....	288	2.4	11.8
Clay-gravel.....	506	1.7	11.8
Heavy clay, wet and sticky.....	83	6.0	12.0
75 per cent loose shale with 25 per cent clay.....	579	3.2	12.4
Heavy clay with a few bowlders.....	101	2.0	12.5
Wet clay with some stumps.....	105	3.2	12.8
Loam with 30 per cent loose rock.....	148	2.1	13.5
Rock, well blasted.....	183	3.4	13.9
Do.....	560	4.2	15.1
Hard shale, well blasted.....	1,434	2.6	16.4
Gneiss, poorly blasted.....	550	10.7	18.5
50 per cent loose rock with 50 per cent unblasted shale rock.....	338	-----	28.0



Average dumping time	
Loamy and sandy soils.....	seconds__ 2.9
Wet and sticky clay.....	do..... 4.0
Rock, well blasted.....	do..... 4.4
Rock, poorly blasted.....	do..... 8.9

FIG. 19.—NUMBER OF DUMPING OPERATIONS PERFORMED IN VARIOUS TIME INTERVALS IN DIFFERENT KINDS OF MATERIAL. BASED ON 10,200 OBSERVATIONS ON 13 DIFFERENT JOBS

especially where the operator is slow or where the mechanism is in poor condition. Because of the general prevalence of insufficient hauling equipment it has become a more or less accepted practice to consider the time required for moving the shovel as of no importance, since it can usually be done while waiting for teams or trucks. This may seem like a good way of neutralizing an inherent shortcoming of the shovel,

(Continued on p. 274)

COMPARATIVE TESTS OF CRUSHED-STONE AND GRAVEL CONCRETE IN NEW JERSEY

REPORT ON COOPERATIVE TESTS CONDUCTED BY THE NEW JERSEY STATE HIGHWAY COMMISSION AND THE U. S. BUREAU OF PUBLIC ROADS

Reported by F. H. JACKSON, Engineer of Tests, Division of Tests, U. S. Bureau of Public Roads

DURING the summer of 1926 the New Jersey State Highway Commission, working in cooperation with the Bureau of Public Roads, conducted a series of concrete tests at the Trenton laboratory of the commission, for the purpose of determining the relative quality and economy of concrete paving mixtures in which 13 different gradations of crushed stone (trap) and gravel were used as coarse aggregate. The investigation involved the making and testing of approximately 150 concrete beams, 8 by 8 by 48 inches in size and 250 concrete cylinders 6 by 12 inches in size.

This report describes in order (1) the various reasons which led up to the investigation, (2) the procedure followed, (3) the results secured, and (4) the conclusions reached, and makes certain recommendations relative to the application of these results. It should be borne in mind that this series of tests was initiated for the purpose of studying the relative merits of two different

types of coarse aggregate produced under certain conditions. The results can therefore be considered as applicable only to the same kinds of material, produced in a similar manner. In order to make possible the drawing of more general conclusions as to the effect of type and gradation of coarse aggregate upon the quality of concrete a series of tests has been started at the Arlington laboratory of the bureau. These tests are

similar to those described here but involve seventeen types of coarse aggregate instead of two. It is anticipated that conclusions suitable for general application will be justified by these tests, but in the meantime those presented here should not be considered as having any application beyond the particular conditions involved.

REASONS LEADING TO INVESTIGATION DISCUSSED

The practice followed in New Jersey and most other States of specifying the same proportions of cement and graded aggregate for concrete, regardless of the void content of these aggregates, has resulted in securing for gravel aggregate an appreciable economic advantage due to the increased yield of concrete obtained from the gravel aggregate on account of its low void content, as well as its lower cost compared with that of crushed stone. These facts, together with the other natural advantages possessed by gravel, such as the increased workability of the resulting concrete, have made its use in general more economical than crushed stone.

Such a condition would not in itself warrant any change in the existing methods of proportioning. It must be remembered, however, that the arbitrary proportions which are set are only a means to an end which, in this case, is the production of concrete possessing certain definite essential physical properties. If a specification is to be considered adequate it must be assumed that, as long as the various details of the specifications relative to materials and construction processes are complied with, concrete of substantially uniform quality will be obtained; in other words, that any variation in either type or gradation of aggregate within the specification limits will not result in any essential change in the quality of the product.

The standard road specifications of the New Jersey Highway Commission require that concrete for pavements shall be mixed in the proportions of 1 part cement to $1\frac{3}{4}$ parts sand and $3\frac{1}{2}$ parts coarse aggregate by

volume measured in a loose, dry condition. In the determination of the field mix, the amount of sand is proportioned on a dry, loose basis; that is, a bulking correction is made. Either crushed rock or gravel conforming to certain requirements as to quality and gradation may be used. This, as has been pointed out, results in the production of more concrete per unit volume of cement when gravel is used as coarse aggregate

than when stone is used, due of course, to the higher void content of the crushed stone aggregate.

It has been repeatedly urged by the crushed-stone interests in New Jersey that concrete produced from crushed stone is of a better quality than that produced from gravel due to the difference in character of these two aggregates and to the higher cement content of the stone concrete, as expressed in terms of volume of cement required to produce a unit volume of concrete.

If these claims are true, they are of considerable significance, because it means that, if the gravel concrete produced under the present specifications is satisfactory in quality, the use of crushed stone results in the production of a higher quality of concrete than is demanded by the minimum requirements of the specifications. If, on the other hand, the crushed-stone concrete is no better than it should be, the obvious conclusion is that the gravel concrete will not meet the minimum requirements. In either event, if it is shown that the type of aggregate does affect the quality of the concrete to an appreciable degree, some readjustment should

THE CONCLUSIONS drawn from these tests should not be interpreted as indicating that crushed stone as a type is superior to gravel as coarse aggregate for cement concrete pavements, but only as indicating that for the particular conditions and kinds of materials involved, concrete in which crushed trap rock was used showed an average flexural strength approximately 12 per cent higher than similar concrete in which gravel was used.

Tests now in progress at the Arlington Laboratory using 17 different coarse aggregates give preliminary indications that materials similar to those used in this investigation will give similar results, but also that the characteristics of the particular aggregate used may be fully as important as the type of material to which it belongs.

be permitted in order to insure concrete of equal quality irrespective of the type of aggregate used.

SCOPE OF TESTS AND MATERIALS DESCRIBED

Recognizing the importance of this problem from both the engineering and economic points of view, the Bureau of Public Roads and the New Jersey State Highway Commission undertook a study of the question through a series of carefully controlled laboratory tests to determine the following:

1. The relative strength and yield of crushed-stone and gravel concrete of the same proportions and consistency, and with the same size and grading of coarse aggregate.

2. What grading of coarse aggregate and what proportions of fine to coarse would give the greatest yield for each type of aggregate, when the concrete is designed for a given strength.

In discussing the essential characteristics of paving concrete, it is herein assumed that in so far as strength is concerned, resistance to bending or flexure is of more significance than is resistance to crushing. Many engineers, in designing concrete pavements, employ the flexural strength of the concrete in calculations for edge and center thickness of slabs under given conditions of load and accept it as the criterion of quality rather than the compressive strength. It is obvious, therefore, that factors influencing flexural strength are of critical importance in so far as pavement concrete is concerned.

To secure flexural-strength tests that would not be unduly influenced by the size of the aggregates used, it was decided to make the major strength comparisons from the results secured with concrete beams 8 by 8 by 48 inches in size, tested as cantilevers for flexural strength using a device similar to that described by Clemmer in Public Roads for May, 1926. It was also decided to use the ends of the beams from the flexure tests as compression specimens in determining the crushing strength of the concrete as well as the 6 by 12 inch cylinders which were cast for this particular purpose. Tests were to be made on all specimens at the age of 28 days, as well as additional series of tests on the 1:1 $\frac{3}{4}$:3 $\frac{1}{2}$ specimens at the age of six months. Accurate measurements of the yield of concrete obtained from each batch were also to be taken.

Thirteen gradations of crushed trap rock and gravel were employed, ranging from a 3-inch maximum size down to a 1-inch maximum size. Well-graded and poorly graded combinations were used, the object being to cover quite a wide range of coarse-aggregate gradations within these limits. The exact screen analysis of each combination is shown in Table 1 and is plotted graphically in Figures 1 and 2, which also show the present New Jersey State requirements for crushed stone for concrete pavements. State requirements for gravel for concrete are similar to those for crushed stone except that more material finer than one-half inch is permitted in the gravel than the crushed stone. For convenience in visualising the gradings of the coarse aggregate, the fineness modulus of each combination was calculated and is shown in Table 2, together with the "maximum size," a value to be used in connection with any design calculations which might be made by the fineness-modulus method. This table also shows the percentage of fine aggregate required by the fineness-modulus method for each

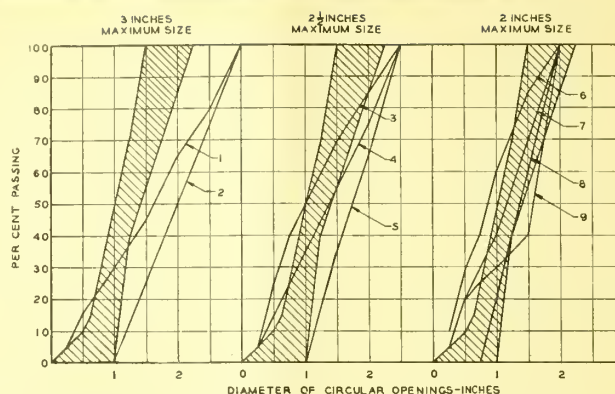


FIG. 1.—RELATION BETWEEN AGGREGATE GRADINGS AND STATE SPECIFICATIONS FOR SIZE OF CRUSHED STONE. AREAS CROSS HATCHED INDICATE SPECIFICATION LIMITS

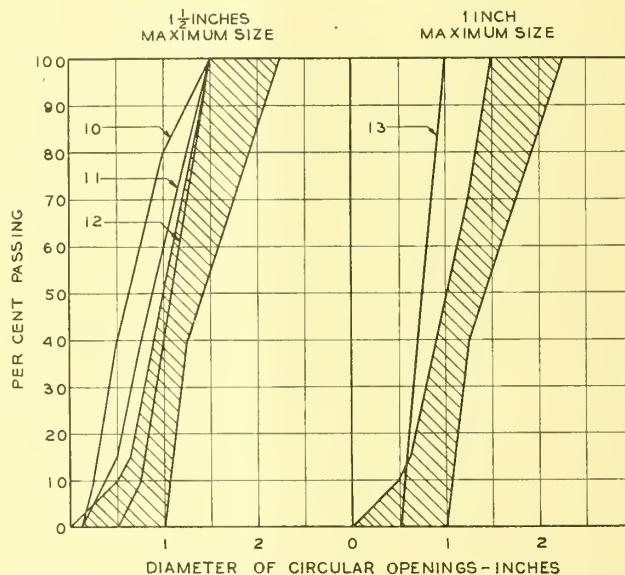


FIG. 2.—RELATION BETWEEN AGGREGATE GRADINGS AND STATE SPECIFICATION FOR SIZE OF CRUSHED STONE. AREAS CROSS HATCHED INDICATE SPECIFICATION LIMITS

TABLE 1.—Gradings of coarse aggregates used in tests

Coarse aggregate grading No.	Total per cent passing screens with round openings								
	3-inch	2 $\frac{1}{2}$ -inch	2-inch	1 $\frac{1}{2}$ -inch	1-inch	$\frac{3}{4}$ -inch	$\frac{1}{2}$ -inch	$\frac{1}{4}$ -inch	$\frac{1}{8}$ -inch
1.....	100	80	65	45	30	-----	15	5	0
2.....	100	75	50	25	0	-----	-----	-----	-----
3.....	-----	100	85	70	50	40	25	5	0
4.....	-----	100	75	55	35	25	15	5	0
5.....	-----	100	65	35	0	-----	-----	-----	-----
6.....	-----	-----	100	85	60	40	30	10	0
7.....	-----	-----	100	70	40	30	20	5	0
8.....	-----	-----	100	60	20	0	-----	-----	-----
9.....	-----	-----	100	45	30	25	20	5	0
10.....	-----	-----	-----	100	80	60	40	10	0
11.....	-----	-----	-----	100	60	40	15	5	0
12.....	-----	-----	-----	100	40	10	0	-----	-----
13.....	-----	-----	-----	-----	100	50	0	-----	-----

gradation calculated on the basis of the sand employed in these tests.

The crushed stone was obtained from Bound Brook, N. J., and the gravel from Morrisville, Pa. The rock is representative of the extensive deposits of basalt (trap rock) quarried in northern New Jersey. This material is very hard and tough, showing a percentage of wear of 2.2 (French coefficient equals 18.2), apparent

specific gravity of 2.97 (weight per cubic foot, solid, 185 pounds), and a water absorption of 0.05 per cent. The gravel is representative of material of the type used extensively throughout this region. It consists essentially of rounded fragments of sandstone, flint, and quartz, has an apparent specific gravity of 2.65 (weight per cubic foot solid, of 165 pounds) and a water absorption of 0.68 per cent.

TABLE 2.—*Fineness moduli of aggregates used in tests*

Coarse aggregate grading No.	Maximum size	Fineness modulus	Sand required ¹
	<i>Inches</i>		<i>Per cent</i>
1.....	3	8.0	31
2.....	3	8.6	39
3.....	3	7.5	30
4.....	3	7.9	30
5.....	3	8.4	37
6.....	1½	7.2	35
7.....	2	7.5	30
8.....	2	8.0	37
9.....	2	7.6	32
10.....	1	6.8	39
11.....	1½	7.3	37
12.....	1½	7.7	42
13.....	1	7.2	45
Sand.....		3.2	-----

¹ Calculated by fineness-modulus method for a designed strength of 3,500 pounds per square inch, slump of 1 inch and with the sand employed in these tests.

The fine aggregate used in the tests was also obtained at Morrisville, Pa., and meets all the conventional tests for first-grade concrete sand. The physical properties of the sand are shown below. Its apparent specific gravity was 2.65 (weight per cubic foot, solid, 165 pounds and its weight per cubic foot, dry, and shaken to refusal 108 pounds).

Physical properties of fine aggregate consisting essentially of subangular quartz grains containing some chert and sandstone

Sieve analysis:

Total retained on ¼-inch screen.....	per cent.....	7
No. 10 sieve.....	do.....	24
No. 20 sieve.....	do.....	38
No. 30 sieve.....	do.....	62
No. 50 sieve.....	do.....	86
No. 100 sieve.....	do.....	95
Silt and clay.....	do.....	1.8
Weight per cubic foot (shaken to refusal).....	pounds.....	108
Tensile strength ratio at 7 days.....	per cent.....	123
28 days.....	do.....	120

The cement was a standard Portland passing all physical test requirements. It was a brand used extensively in New Jersey.

TABLE 3.—*Weight per cubic foot and percentage of voids of coarse aggregates*

Coarse aggregate grading No.	Crushed stone		Gravel	
	Weight per cubic foot	Voids	Weight per cubic foot	Voids
	<i>Pounds</i>	<i>Per cent</i>	<i>Pounds</i>	<i>Per cent</i>
1.....	110	41	108	34
2.....	97	48	97	41
3.....	106	43	107	35
4.....	103	44	108	34
5.....	94	49	96	42
6.....	102	45	106	36
7.....	103	44	105	36
8.....	97	48	97	41
9.....	104	44	107	35
10.....	97	48	105	36
11.....	98	47	104	37
12.....	94	49	98	41
13.....	90	51	96	42

The percentage of voids for each aggregate in each of the 13 grading combinations as calculated from the

apparent specific gravities and the unit weights are shown in Table 3. These values were used in all calculations of volume-weight relations.

TEST PROCEDURE FOR SERIES A

In series A the concrete was proportioned by volume, using a nominal mix of 1:1¾:3½. As far as could be measured by the use of the flow table, a constant consistency was maintained throughout the series, the quantity of water being varied as necessary. The flow test was made on the 30-inch flow table, using 30 drops from a height of one-half inch. In this report the flow is expressed as the final diameter in tenths of inches of a truncated cone of concrete, which had an original diameter at the base of 10 inches. For example, a flow of 140 indicates a final diameter of the base of 14 inches. The consistency employed, which was as nearly as possible that required in actual construction, gave a flow of 140 corresponding to a slump of from 1 inch to 2 inches. It was found necessary to vary the water-cement ratio from 0.67 to 0.83 in order to maintain a constant consistency throughout the range of coarse-aggregate gradings. The water-cement ratios used in series A are given in Table 4. No correction for water absorbed by the aggregate was made in calculating these values.

TABLE 4.—*Water-cement ratio used in series A to produce concrete of uniform consistency*

[Proportions: 1:1¾:3½ by volume. Consistency: Flow equals approximately 140]

Coarse aggregate grading No.	Water-cement ratio for stone concrete	Water-cement ratio for gravel concrete	Coarse aggregate grading No.	Water-cement ratio for stone concrete	Water-cement ratio for gravel concrete
1.....	0.76	0.76	8.....	0.71	0.74
2.....	.69	.70	9.....	.76	.74
3.....	.76	.77	10.....	.83	.78
4.....	.72	.70	11.....	.74	.74
5.....	.68	.66	12.....	.73	.70
6.....	.76	.75	13.....	.79	.74
7.....	.74	.72			

The actual quantities of materials required for each batch of concrete were determined by weight, using the volume-weight relations given in Table 3. A sufficient quantity of each size of aggregate was weighed for each batch, so as to secure the required total volume. In this way, all tendency toward segregation was avoided. In order to properly gauge the quantity of water required for each test batch, a trial batch of the same proportions was mixed by hand and the proper amount of water determined by trial. This procedure, while adding considerably to the labor involved, resulted in test batches of much greater uniformity in consistency than would otherwise have been possible.

The actual test batches were mixed in a 1-bag gasoline-driven mixer. For the gradings numbered 1 to 5, inclusive, the quantities of materials used in each batch were based on a coarse aggregate volume of 1.8 cubic feet, producing a slight excess of concrete over that required for one beam. For the gradings numbered 6 to 13, inclusive, quantities were based on a coarse-aggregate volume of 2.4 cubic feet, producing sufficient concrete for one beam and three cylinders. All batches were mixed for two minutes after all ingredients had been added. After mixing, the batch was dumped upon a water-tight platform, flow tests

for consistency made, and specimens cast. Concrete was rodded into molds in accordance with approved methods of molding. Accurate determinations of yield were made by determining both the volume and the weight of the concrete remaining after the specimens had been cast.

Specimens were cured for 24 hours under damp cloths in the laboratory, after which the forms were removed and the specimens placed in a moist room until tested. Tests were made at 28 days and at 6 months.

To eliminate the effect of other variables on the relative strength of the crushed-stone and gravel concrete, specimens employing crushed stone and gravel of the same gradation were always cast in pairs. The entire series of gradations was also completed before being repeated, several days usually elapsing between the first and second rounds. Variations in

temperature and humidity conditions throughout the test were compensated for in this way, making possible a more satisfactory comparison of the effect of aggregate grading on strength.

In testing the beams for flexural strength, two breaks were made on each specimen and the results averaged and reported, after calculation, as the modulus of rupture for the round. (See Table 5.) The corresponding values for round 2 were obtained in the same way. The average of the two rounds was reported as the average value for the type of aggregate, the grading, and the age involved. Each average value for modulus of rupture represents four breaks on two specimens tested on different days.

Average compression test values, as given in Table 6, were obtained in the same manner, except those for gradings Nos. 6 to 13, inclusive, each value for each round is the average of tests on three cylinders.

TABLE 5.—Flexural strength of concrete specimens of series A mixed in the proportions $1:1\frac{3}{4}:3\frac{1}{2}$ by volume (nominal mix) and with a flow of approximately 140¹

Coarse aggregate grading No.	Modulus of rupture, 28-day tests						Modulus of rupture, 6-month tests					
	Stone			Gravel			Stone			Gravel		
	Round No. 1	Round No. 2	Average	Round No. 1	Round No. 2	Average	Round No. 1	Round No. 2	Average	Round No. 1	Round No. 2	Average
	Lbs. per sq. in.	Lbs. per sq. in.	Lbs. per sq. in.	Lbs. per sq. in.	Lbs. per sq. in.	Lbs. per sq. in.	Lbs. per sq. in.	Lbs. per sq. in.	Lbs. per sq. in.	Lbs. per sq. in.	Lbs. per sq. in.	Lbs. per sq. in.
1.....	545	565	555	475	530	500	610	670	640	590	585	590
2.....	535	515	525	470	455	460	610	590	600	575	590	580
3.....	620	575	600	455	510	480	630	655	640	580	560	570
4.....	600	615	605	520	530	525	680	650	665	605	600	600
5.....	540	625	580	530	555	540	675	705	690	610	590	600
6.....	550	555	550	455	490	470	705	770	740	625	555	590
7.....	505	465	485	435	480	460	680	655	670	635	555	595
8.....	515	515	515	435	475	455	770	660	715	655	560	610
9.....	550	630	590	490	605	550	700	780	740	695	675	685
10.....	575	610	590	540	555	550	605	735	670	595	640	620
11.....	690	610	650	560	490	525	670	645	660	645	550	600
12.....	525	585	555	520	465	490	675	685	680	580	555	570
13.....	555	575	565	565	535	550	690	720	705	630	580	605
Average.....			570			505			680			600
Maximum.....			650			550			740			685
Minimum.....			485			455			600			570

¹ Determinations made on concrete beams 8 by 8 by 48 inches in size, tested as cantilevers. Each value is the average of two breaks on one beam.

TABLE 6.—Compressive strength of concrete specimens of series A mixed in the proportions $1:1\frac{3}{4}:3\frac{1}{2}$ by volume (nominal mix) and with a flow of approximately 140¹

Coarse aggregate grading No.	Crushing strength, 28-day tests						Crushing strength, 6-month tests					
	Stone			Gravel			Stone			Gravel		
	Round No. 1	Round No. 2	Average	Round No. 1	Round No. 2	Average	Round No. 1	Round No. 2	Average	Round No. 1	Round No. 2	Average
	Lbs. per sq. in.	Lbs. per sq. in.	Lbs. per sq. in.	Lbs. per sq. in.	Lbs. per sq. in.	Lbs. per sq. in.	Lbs. per sq. in.	Lbs. per sq. in.	Lbs. per sq. in.	Lbs. per sq. in.	Lbs. per sq. in.	Lbs. per sq. in.
1.....	4,255	3,160	3,710	3,580	3,020	3,300						
2.....	3,955	3,050	3,500	3,600	3,455	3,530						
3.....	4,300	3,385	3,840	3,595	3,565	3,580						
4.....	3,630	3,470	3,550	3,255	3,440	3,350						
5.....	3,510	3,760	3,635	3,475	3,370	3,420						
6.....	3,285	2,860	3,070	3,070	2,890	2,980	4,630	5,085	4,860	3,345	4,700	4,020
7.....	3,145	2,920	3,030	3,115	3,360	3,240	4,050	4,205	4,130	4,090	4,145	4,120
8.....	3,230	2,720	2,975	2,740	2,685	2,710	4,535	3,405	3,970	4,420	3,110	3,765
9.....	3,215	3,505	3,360	2,860	3,510	3,185	3,880	4,090	3,985	3,740	4,570	4,155
10.....	3,830	3,020	3,425	3,620	3,600	3,610	4,695	3,945	4,320	4,220	4,525	4,370
11.....	3,715	3,285	3,500	3,635	3,490	3,560	4,780	3,295	4,040	4,610	3,655	4,130
12.....	3,530	2,410	2,970	3,540	3,070	3,305	4,420	3,725	4,070	4,205	4,305	4,255
13.....	3,540	2,890	3,215	3,600	3,725	3,660	4,030	3,595	3,810	4,495	4,695	4,595
Average.....			3,370			3,340			4,150			4,180
Maximum.....			3,840			3,660			4,860			4,595
Minimum.....			2,970			2,710			3,810			3,765

¹ For coarse aggregate gradings Nos. 1 to 5, inclusive, crushing-strength tests were made on portions of beams from flexure tests; each value is the average of two tests. For coarse aggregate gradings Nos. 6 to 13, inclusive, crushing-strength tests were made on 6 by 12 inch cylinders; each value is the average of three tests.

TEST PROCEDURE FOR SERIES B

In series B an entirely different method of proportioning was employed, but the various details of mixing, molding, curing, and testing were exactly the same as in series A. In this series the concrete was designed according to the so-called trial method application of the water-cement ratio theory which is being advocated by the Portland Cement Association and which is described in a bulletin¹ published by the association. The design called for a crushing strength at 28 days of 3,500 pounds, using a water-cement ratio of 0.72. The object was to secure data showing how variations in the type and gradation of coarse aggregate and the ratio of fine to coarse aggregate would affect the strength and yield when proportioned by the trial method.

To secure a uniform consistency as used in series A but with a constant water-cement ratio, the procedure in determining the proper proportions was as follows:

A paste of water and cement in the ratio required (0.72) was prepared and added to a mixture of fine and coarse aggregate until a consistency was obtained approximating a flow of 140. Three mixtures of fine and coarse aggregate were used. They were proportioned by volume in the following ratios, 33:67, 36:64, and 40:60. Trials were made with small hand-mixed batches for each type and grading of coarse aggregate, using each of the ratios of fine to coarse aggregate, and the proportions for the test batches were calculated from the resulting weights. The calculated proportions in series B for all combinations are given in Table 7. This work was done in strict accordance with the procedure recommended by the Portland Cement Association for the design of concrete mixtures.

There were slight differences in consistency between the machine-mixed batches and the small trial batches, even when gauged with the same amount of water, and some preliminary experimental work was necessary to establish the relation between the two. After a little experience it was found possible to proportion the trial batches for a water-cement ratio of 0.72, so that the consistency of the large test batches after two minutes mixing would not vary more than ± 5 points from a flow of 140. Any batches showing a greater deviation were discarded.

The necessity of maintaining a uniform consistency was recognized. It is realized, of course, that there is an intimate relation between consistency, water-cement ratio, and proportions for different gradations of aggregate. If any two of these factors are kept constant, it is necessary to vary the other whenever the aggregate grading varies. In this case only the coarse aggregate varied and the sand remained constant. In series A the proportions and consistency were kept constant and the water-cement ratio varied, while in series B the consistency and water-cement ratio were kept constant and the proportions varied. The latter is, of course, the rational method if we assume that strength is proportional to water-cement ratio within the range of workable mixtures.

STRENGTH DATA OBTAINED IN SERIES A DISCUSSED

As previously stated, the tests included in series A were made to obtain some definite information as to the relative strength and yield of crushed-stone and

gravel concrete when proportioned in accordance with current New Jersey specifications and with the various coarse-aggregate gradings indicated.

The results of the flexure tests at both 28 days and 6 months are given in Table 5 and are plotted in Figure 3. An examination of this table indicates that the crushed-stone concrete is considerably higher in flexural strength than the gravel concrete. The average increase for all 13 gradations is seen to be 65 pounds per square inch at 28 days, and 80 pounds per square inch at 6 months; approximately 13 per cent in both cases. The concordance of the results is quite remarkable in view of the rather limited number of test specimens involved. Out of a total of 52 pairs of test specimens in the group, the crushed-stone concrete shows higher flexural strength in 49 cases; the values are the same in one case and the gravel concrete shows higher values in two cases. An inspection of the averages of the two rounds shows the crushed-stone concrete higher in all cases, as is indicated graphically in Figure 3. The results given in Table 6 and plotted in Figure 4, however, indicate that this difference is not reflected in the crushing strength of the concrete, the crushed-stone concrete in this case averaging practically the same as the gravel concrete at each period. These tables and figures do not reveal any consistent relation between either maximum size or gradation

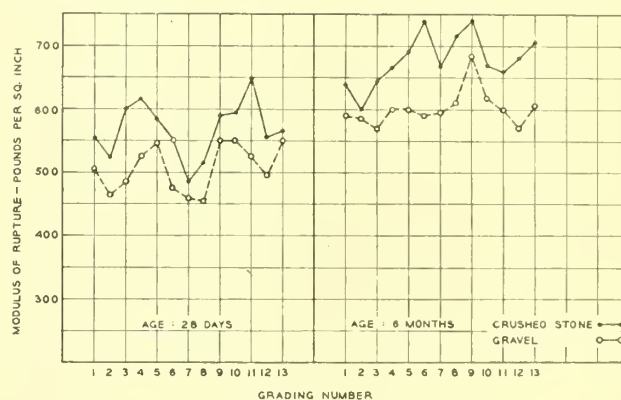


FIG. 3.—RESULTS OF FLEXURE TESTS OF CONCRETE SPECIMENS OF SERIES A

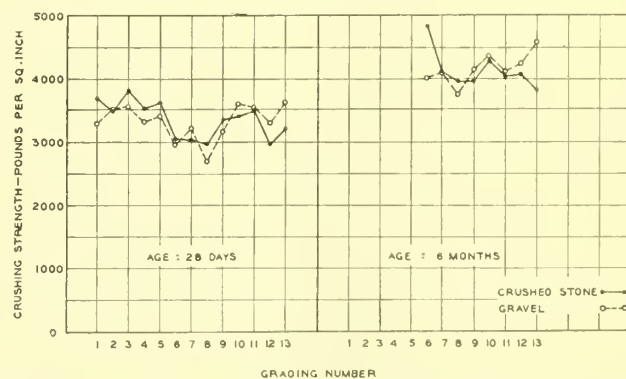


FIG. 4.—RESULTS OF CRUSHING STRENGTH TESTS OF CONCRETE SPECIMENS OF SERIES A

of coarse aggregate and the strength, either flexural or crushing. This is not particularly surprising in view of the small number of specimens of each gradation represented. This would seem to indicate that possibly variations in grading do not have a very marked effect upon strength, at least in so far as concrete of

¹ DESIGN AND CONTROL OF CONCRETE MIXTURES, published by the Portland Cement Association, Chicago, Ill.

this particular proportion ($1 : 1\frac{3}{4} : 3\frac{1}{2}$) is concerned. It is true that a number of the mixes were undersanded when judged by the standard set-up by the fineness-modulus method of proportioning. Gradings Nos. 2, 5, 8, 10, 11, 12, and 13 when proportioned by this method would require a sand-coarse aggregate ratio of approximately 2 : 3 instead of 1 : 2 as given by the $1 : 1\frac{3}{4} : 3\frac{1}{2}$ mix.

In the case of the smaller aggregates, as for instance gradings Nos. 10 to 13, inclusive, it is probable that this additional sand would have necessitated sufficient additional water for the same consistency to cause an appreciable lowering of the strength. Variations in the water-cement ratio required because of variations in coarse-aggregate grading with a fixed quantity of sand, as used in this series (Table 4) do not appear to be sufficient to affect the strength consistently. This fact is of considerable practical significance because it indicates that changes in the water content made necessary on the job to maintain a given consistency when variations in coarse-aggregate grading occur do not affect the strength as much as is sometimes supposed. There is, however, a very appreciable effect on yield, and therefore on relative economy, which will be discussed later.

STRENGTH DATA OBTAINED IN SERIES B DISCUSSED

The data secured as the result of tests of series B are interesting because we have not only an opportunity to study variation in strength for a fixed water-cement ratio, but we can also study the effect of variation in the ratio of sand to coarse aggregate on the strength and yield. It will be remembered that in this series the concrete was proportioned by trial so as to give a consistency of 140 by the flow test for a water-cement ratio of 0.72 and that three sand-coarse-aggregate ratios were employed, 33 : 67, 36 : 64, and 40 : 60. The actual proportions which were derived are given in Table 7, and vary all the way from a 1 : 1.67 : 2.50 mix for grading No. 10, stone (40 : 60 ratio), to a 1 : 1.98 : 3.96 mix for grading No. 2, gravel (33 : 67 ratio). The resulting yield and therefore economy of these mixes will be discussed

later. These mixes were all designed to give a crushing strength at 28 days of 3,500 pounds per square inch.

TABLE 7.—Proportions used in series B to obtain flow of 140 with water-cement ratio of 0.72

Coarse aggregate grading No.	Ratio sand to coarse aggregate 33:67		Ratio sand to coarse aggregate 36:64		Ratio sand to coarse aggregate 40:60	
	Stone	Gravel	Stone	Gravel	Stone	Gravel
1.....	1:1.68:3.37	1:1.72:3.44	1:1.72:3.06	1:1.81:3.22	1:1.78:2.67	1:1.85:2.78
2.....	1:1.96:3.92	1:1.98:3.96	1:1.96:3.49	1:2.01:3.58	1:2.05:3.07	1:2.14:3.21
3.....	1:1.62:3.24	1:1.68:3.36	1:1.70:3.08	1:1.82:3.24	1:1.86:2.79	1:1.92:2.88
4.....	1:1.79:3.58	1:1.63:3.27	1:1.84:3.28	1:1.85:3.29	1:1.93:2.90	1:1.99:2.99
5.....	1:1.88:3.75	1:1.90:3.80	1:1.97:3.51	1:1.99:3.55	1:1.92:2.88	1:2.18:3.28
6.....	1:1.52:3.04	1:1.55:3.11	1:1.63:2.90	1:1.66:2.95	1:1.84:2.76	1:1.88:2.82
7.....	1:1.57:3.14	1:1.70:3.39	1:1.80:3.20	1:1.73:3.08	1:1.82:2.73	1:1.88:2.82
8.....	1:1.84:3.68	1:1.87:3.75	1:1.93:3.44	1:1.95:3.47	1:2.02:3.03	1:2.01:3.02
9.....	1:1.67:3.34	1:1.71:3.42	1:1.90:3.38	1:1.81:3.22	1:1.78:2.68	1:1.89:2.83
10.....	1:1.47:2.94	1:1.47:2.94	1:1.53:2.72	1:1.52:2.71	1:1.67:2.50	1:1.78:2.68
11.....	1:1.59:3.18	1:1.62:3.24	1:1.70:3.03	1:1.72:3.06	1:1.91:2.87	1:1.83:2.74
12.....	1:1.72:3.44	1:1.82:3.64	1:1.90:3.38	1:1.80:3.20	1:2.06:3.09	1:2.03:3.05
13.....	1:1.62:3.24	1:1.67:3.35	1:1.74:3.10	1:1.71:3.05	1:1.91:2.86	1:1.93:2.90

The results of the flexural strength test for each of the three aggregate ratios are shown in Table 8 and are plotted in Figure 5. Inspecting general averages, not only were almost identical values for moduli of rupture at 28 days found as were found for series A, but what is somewhat more significant, the same relative increase in strength for the crushed-stone concrete as compared with the gravel concrete was found. These average values are grouped together for comparison in Table 9 and Figure 6. The percentage of increase, is however, not quite so great, ranging from 10 per cent for the 33:67 mix to 13 per cent for the 40:60 combination.

The figures demonstrate the danger of designing concrete for resistance to flexure solely on the basis of water-cement ratio without regard to the type of aggregate employed.

Reverting again to Figure 5 and discussing the detail values for flexural strength in series B, it is seen that the graphs show considerable variation in strength. Attention should be called, however, to the fact that the vertical scale to which these values have been plotted is very large and that in reality the deviations from the group averages are in general quite small.

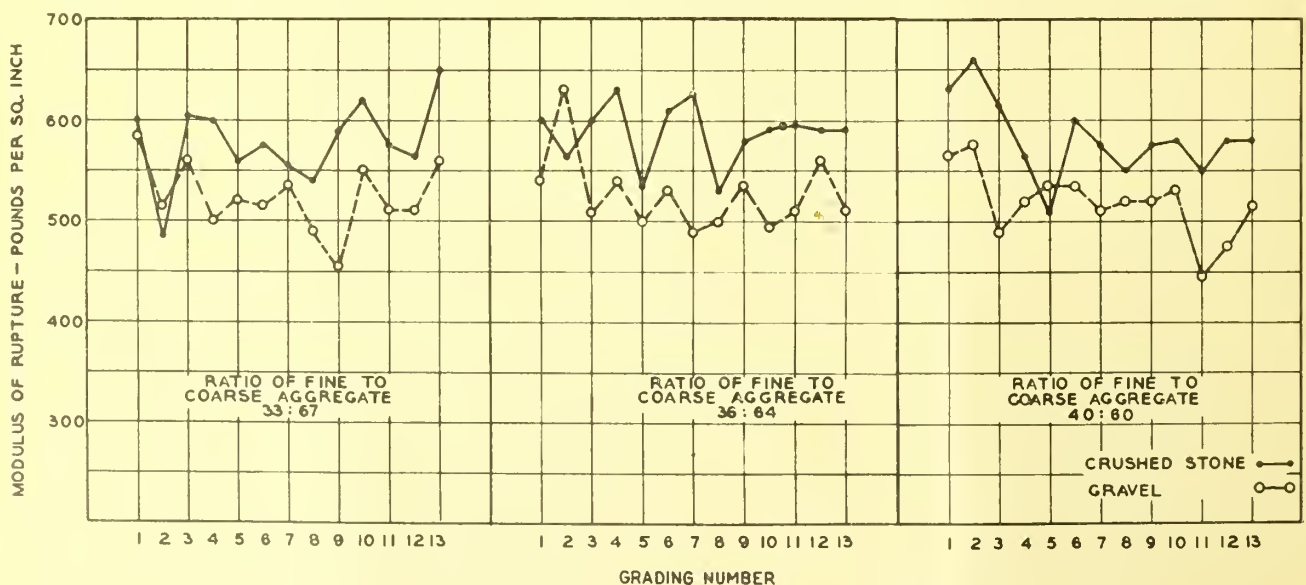


FIG. 5.—RESULTS OF FLEXURE TESTS OF CONCRETE SPECIMENS OF SERIES B, 28-DAY TESTS

TABLE 8.—Flexural strength of concrete specimens of series B, 28-day tests ¹

Coarse aggregate grading No.	Ratio sand to coarse aggregate 33:67						Ratio sand to coarse aggregate 36:64						Ratio sand to coarse aggregate 40:60					
	Stone			Gravel			Stone			Gravel			Stone			Gravel		
	Round No. 1	Round No. 2	Average	Round No. 1	Round No. 2	Average	Round No. 1	Round No. 2	Average	Round No. 1	Round No. 2	Average	Round No. 1	Round No. 2	Average	Round No. 1	Round No. 2	Average
	Lbs. per sq. in.	Lbs. per sq. in.	Lbs. per sq. in.	Lbs. per sq. in.	Lbs. per sq. in.	Lbs. per sq. in.	Lbs. per sq. in.	Lbs. per sq. in.	Lbs. per sq. in.	Lbs. per sq. in.	Lbs. per sq. in.	Lbs. per sq. in.	Lbs. per sq. in.	Lbs. per sq. in.	Lbs. per sq. in.	Lbs. per sq. in.	Lbs. per sq. in.	Lbs. per sq. in.
1	660	540	600	640	530	585	615	585	600	525	555	540	690	565	630	575	550	560
2	520	450	485	515	515	515	565	560	560	570	685	630	720	595	660	625	520	570
3	570	640	605	575	540	560	560	635	600	510	505	510	585	645	615	475	505	490
4	580	620	600	505	500	500	605	655	630	525	550	540	555	570	560	525	510	520
5	590	525	560	605	530	520	560	505	530	480	525	500	495	525	510	545	520	530
6	590	555	570	530	500	515	580	640	610	500	560	530	600	605	600	530	535	530
7	565	545	555	555	510	530	665	590	630	515	465	490	565	585	575	470	550	510
8	525	550	540	490	485	490	585	475	530	525	480	500	550	545	550	490	545	520
9	575	605	590	510	400	455	600	555	580	540	525	530	575	575	575	545	490	520
10	600	645	620	505	590	550	600	580	590	510	475	490	580	575	580	540	520	530
11	595	555	575	515	500	510	600	590	595	480	535	510	575	530	550	420	465	440
12	560	570	565	515	500	510	575	600	590	545	570	560	620	585	580	490	455	470
13	665	630	650	530	585	560	595	585	590	510	510	510	595	565	580	495	535	515
Average			580			525			590			525			580			515
Maximum			650			585			630			630			660			570
Minimum			485			455			530			490			510			440

¹ Determinations made on concrete beams 8 by 8 by 48 inches in size. Each value is the average of two breaks on one beam.

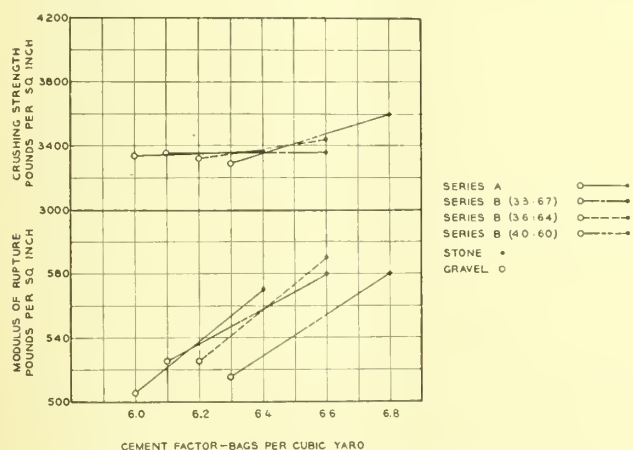


FIG. 6.—RELATION BETWEEN STRENGTH OF CONCRETE AND CEMENT FACTOR. GENERAL AVERAGES FOR ALL GRADINGS

TABLE 9.—Comparison of average results of 28-day tests of series A and series B

Series	Coarse aggregate	Ratio of fine to coarse aggregate	Average modulus of rupture	Average crushing strength	Average cement factor
			Lbs. per sq. in.	Lbs. per sq. in.	Bags per cu. yd.
A	Stone	33:67	570	3,370	6.4
A	Gravel	33:67	505	3,340	6.0
B	Stone	33:67	580	3,360	6.6
B	Gravel	33:67	525	3,360	6.1
B	Stone	36:64	590	3,440	6.6
B	Gravel	36:64	525	3,310	6.2
B	Stone	40:60	580	3,600	6.8
B	Gravel	40:60	515	3,290	6.3

NOTE.—Each value for flexural strength is the average of 52 tests. Each value for crushing strength is the average of 68 tests. In series A the proportions were 1:1½:3½ by volume; flow, 140; water-cement ratio varied from 0.67 to 0.81. In series B the proportions were determined by trial (Portland Cement Association method to give a flow of 140, with a water-cement ratio of 0.72).

Table 10 shows the deviations in percentage of the individual values for each grading and type of aggregate from the group average. It will be seen that the average deviations for the three groups in series B are essentially the same and are slightly less than those in series A. It is interesting to note also that in six cases out of eight, grading No. 2 shows a deviation in

excess of 5 per cent. This was a poorly graded material with a maximum size of 3 inches. The most significant detail as regards these figures, however, is the fact that for all intents and purposes the flexural strengths obtained in series B for each type of aggregate are practically uniform, the deviation being no more than must be expected even in carefully conducted laboratory work.

TABLE 10.—Deviations in per cent of individual values for flexural strength from the mean of each group and for each material, based on 28-day tests ¹

Coarse aggregate grading No.	Series A		Series B							
	Stone	Gravel	Ratio fine to coarse aggregate, 33:67		Ratio fine to coarse aggregate, 36:64		Ratio fine to coarse aggregate, 40:60			
			Stone	Gravel	Stone	Gravel	Stone	Gravel	Stone	Gravel
	Per ct.	Per ct.	Per ct.	Per ct.	Per ct.	Per ct.	Per ct.	Per ct.	Per ct.	Per ct.
1	-3	-1	+3	+11	+2	+3	+9	+9	+9	+9
2	-8	-9	-16	-2	-5	+20	+14	+11		
3	+5	-5	+4	+7	+2	-3	+6	-5		
4	+8	+4	+3	-5	+7	+3	-3	+1		
5	+2	+7	-3	-1	-10	-5	-12	+3		
6	-4	-7	-2	-2	+3	+1	+3	+3		
7	-15	-9	-4	+1	+7	-7	-1	-1		
8	-10	-10	-7	-7	-10	-5	-5	+1		
9	+4	+9	+2	-13	-2	+1	-1	+1		
10	+4	+9	+7	+5	0	-7	0	+3		
11	+14	+4	-1	-3	+1	-3	-5	-15		
12	-3	-3	-3	-3	0	+7	0	-9		
13	-1	+9	+12	+7	0	-3	0	0		
Average deviation	6	7	5	5	4	5	5	5		

¹ Total number of values, 104. Number in which deviation exceeds 10 per cent, 11; number in which deviation exceeds 5 per cent, 35; number in which deviation is 5 per cent or less, 66.

The results of crushing strength tests on specimens of series B are shown in Table 11 and are plotted in Figure 7. The group averages are summarized in Table 9. As in the case of the flexure tests, the deviation from the averages are, in general, neither consistent nor of great magnitude. Either of the three aggregate ratios seem to produce concrete of about the same strength regardless of grading, which again seems to indicate that so far as strength alone is concerned grading of coarse aggregate within the limits here indicated is not an important factor. The words "strength alone" are used here and elsewhere advisedly.

Aside from the yield or relative economy of the different gradations, which is a subject entirely apart from the question of quality, variations in grading which produce harsh concrete difficult to handle and place, will unquestionably affect the strength on the job, because of the invariable tendency of the concrete gang to use excess water with such a mix. In these tests an effort was made to secure comparable workability of the different batches by means of the flow test. The slump test was, of course, entirely unsuited to this kind of work. The flow test was used because there was nothing better. It is, however, not a test for workability in the strict sense of the word, so that several of the undersanded mixtures with the ratio of fine to coarse aggregate of 33:67 were probably unworkable in the sense that they would have been

TABLE 11.—Results of crushing strength tests of specimens of series B at age of 28 days¹

RATIO OF FINE TO COARSE AGGREGATE 33:67

Coarse aggregate grading No.	Crushed stone			Gravel		
	Round No. 1	Round No. 2	Average	Round No. 1	Round No. 2	Average
	Lbs. per sq. in.	Lbs. per sq. in.	Lbs. per sq. in.	Lbs. per sq. in.	Lbs. per sq. in.	Lbs. per sq. in.
1.....	3,370	3,215	3,290	3,230	3,595	3,410
2.....	3,370	3,040	3,205	3,220	3,185	3,200
3.....	4,230	3,860	4,045	3,785	3,855	3,820
4.....	3,825	4,065	3,945	3,690	3,240	3,465
5.....	3,590	3,150	3,370	3,820	2,605	3,210
6.....	3,710	3,205	3,460	3,345	4,075	3,710
7.....	3,455	3,145	3,300	3,375	3,410	3,390
8.....	2,950	2,645	2,800	3,200	3,210	3,205
9.....	3,695	2,985	3,340	3,365	3,720	3,540
10.....	4,025	3,090	3,560	3,490	3,405	3,450
11.....	3,710	3,000	3,355	3,635	2,580	3,210
12.....	3,025	2,580	2,800	3,120	2,535	2,830
13.....	3,465	2,945	3,205	3,205	3,315	3,260
Average.....			3,360			3,360
Maximum.....			4,045			3,820
Minimum.....			2,800			2,830

RATIO OF FINE TO COARSE AGGREGATE 36:64

1.....	3,520	3,245	3,380	3,605	3,385	3,495
2.....	3,090	3,075	3,080	3,245	4,015	3,630
3.....	3,975	3,500	3,770	3,165	3,160	3,160
4.....	3,645	4,060	3,850	3,190	3,325	3,260
5.....	3,455	3,375	3,415	2,975	3,685	3,330
6.....	3,790	3,805	3,800	3,190	3,815	3,500
7.....	3,285	3,925	3,605	3,085	3,215	3,150
8.....	2,845	2,895	2,870	2,885	2,725	2,805
9.....	3,125	3,190	3,160	3,170	3,000	3,085
10.....	3,550	3,950	3,750	3,640	3,680	3,660
11.....	3,175	3,785	3,480	3,225	3,935	3,580
12.....	3,025	3,310	3,170	3,440	2,990	3,215
13.....	3,225	3,580	3,400	2,870	3,515	3,190
Average.....			3,440			3,310
Maximum.....			3,850			3,660
Minimum.....			2,870			2,805

RATIO OF FINE TO COARSE AGGREGATE 40:60

1.....	3,970	3,360	3,665	3,040	3,460	3,250
2.....	3,335	3,805	3,570	3,490	3,425	3,460
3.....	3,765	4,065	3,915	3,045	3,890	3,470
4.....	3,670	4,190	3,930	3,240	2,895	3,070
5.....	3,500	3,590	3,545	3,365	3,150	3,260
6.....	4,165	3,385	3,775	3,490	3,460	3,475
7.....	3,540	2,865	3,200	3,180	3,630	3,405
8.....	3,200	3,060	3,130	3,285	3,115	3,200
9.....	3,720	3,475	3,600	3,190	2,970	3,080
10.....	4,055	3,920	3,990	3,610	3,260	3,435
11.....	3,610	3,600	3,605	3,480	3,285	3,380
12.....	3,145	3,235	3,190	3,020	2,625	2,820
13.....	3,435	3,905	3,670	3,540	3,365	3,450
Average.....			3,600			3,290
Maximum.....			3,990			3,475
Minimum.....			3,130			2,820

¹ For coarse-aggregate gradings 1 to 5, inclusive, crushing-strength tests were made on portion of beams from flexure tests. Each value is the average of two breaks. For coarse-aggregate gradings 6 to 13, inclusive, crushing-strength tests were made on 6 by 12 inch cylinders. Each value is average of three tests.

difficult to place and finish on the job. The point that it is intended to emphasize is, that it was possible to control the water-cement ratio in the laboratory but it would not be at all easy in actual construction unless the control was very rigid.

ABSORPTION TESTS MADE

A short series of absorption tests was made on the concrete of series A at age of six months. The results are given in Table 12 and indicate that there is practically no difference between the absorption of the stone and the gravel concrete. These tests were made by immersing samples of concrete from the broken beams in water for 24 hours and noting the increase in weight.

TABLE 12.—Absorption tests on concrete of series A at age of six months

Coarse aggregate grading No.	Water absorbed by stone concrete			Water absorbed by gravel concrete		
	Specimen No. 1	Specimen No. 2	Average	Specimen No. 1	Specimen No. 2	Average
	Per cent	Per cent	Per cent	Per cent	Per cent	Per cent
1.....	5.5	5.0	5.3	4.8	5.9	5.4
2.....	5.8	4.8	5.3	4.8	5.2	5.0
3.....	5.9	5.6	5.8	5.6	5.5	5.6
4.....	4.3	5.1	4.7	4.5	5.0	4.8
5.....	4.3	5.6	5.0	4.9	5.0	5.0
6.....	5.6	5.6	5.6	5.6	5.8	5.7
7.....	5.6	5.6	5.6	5.1	4.9	5.0
8.....	5.0	6.3	5.7	5.0	5.4	5.2
9.....	6.0	5.8	5.9	4.8	5.2	5.0
10.....	6.2	5.8	6.0	5.1	6.0	5.6
11.....	5.2	5.7	5.5	4.8	5.9	5.4
12.....	4.9	5.0	5.0	5.2	5.8	5.5
13.....	5.4	5.8	5.6	5.6	5.5	5.6
Average.....			5.5			5.3

YIELD OF CONCRETE DISCUSSED

A study of the relative economy of various concrete mixtures, all of which may have the same strength, is just beginning to occupy the attention of engineers and progressive contractors. It was formerly assumed and is still stated in many handbooks that, for a given proportion of cement, sand, and coarse aggregate, definite amounts of materials are required to produce a cubic yard of concrete. While the values given may be correct as average values it is known that there are many factors which may appreciably alter any one or more of these quantities under certain conditions. In this report the writer is concerned primarily with the effect of type and gradation of coarse aggregate on yield.

SERIES A

Table 13 gives the computed quantities of material required for 1 cubic yard of concrete for each type and gradation of aggregate used in these tests, while Table 14 gives the corresponding solid volumes of the materials and the density of the concrete (total solids expressed as a percentage of 27 cubic feet). Quantities of materials are plotted in Figures 8 and 9. These figures illustrate graphically the fluctuations in the quantities of materials required to produce 1 cubic yard of concrete due to variations in the grading of the coarse aggregate. Assuming unit costs for cement, sand, and coarse aggregate, these values may be used to determine the cost of the materials in a cubic yard of concrete for each grading and aggregate type shown. Allowance should be made for the fact that the figures for any particular case may be somewhat out of line because of inaccuracies which result from experimental

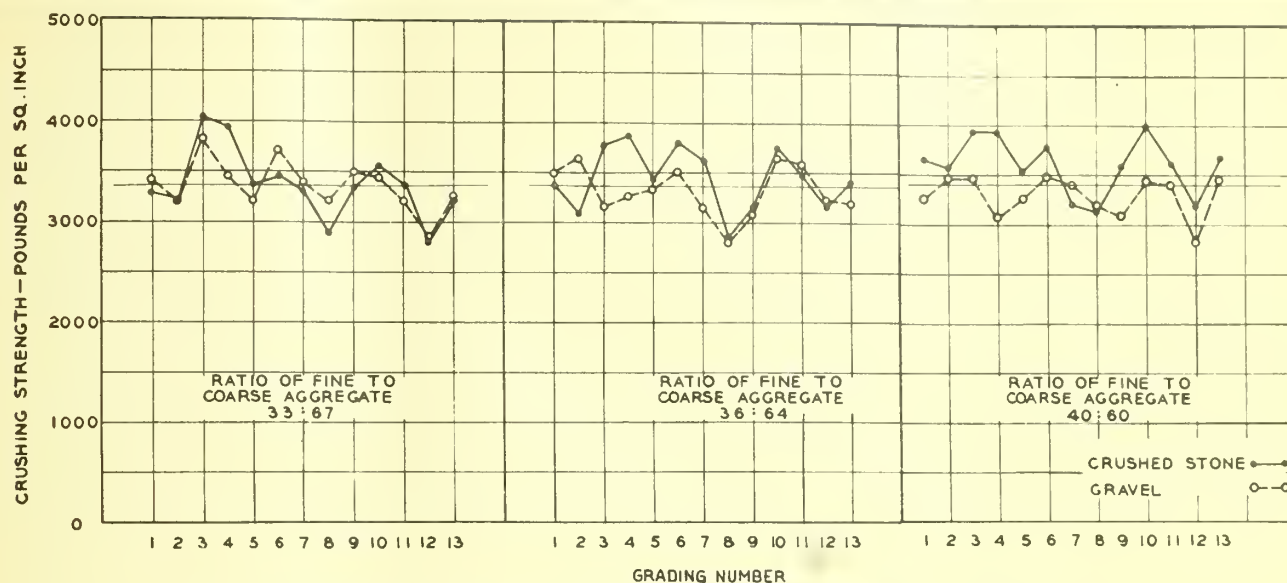


FIG. 7.—CRUSHING STRENGTH OF CONCRETE SPECIMENS OF SERIES B AT AGE OF 28 DAYS

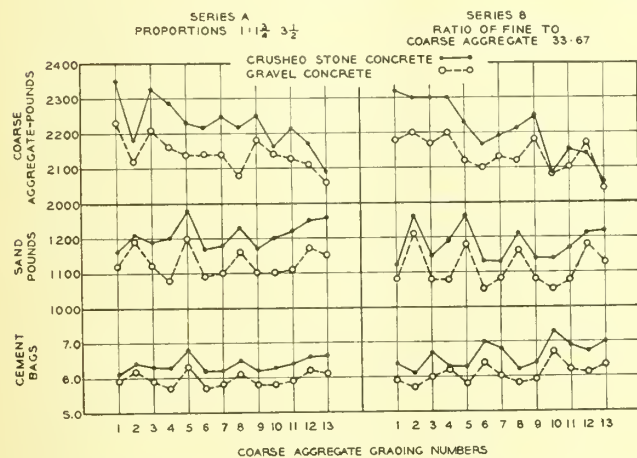


FIG. 8.—QUANTITIES OF MATERIALS REQUIRED FOR 1 CUBIC YARD OF CONCRETE

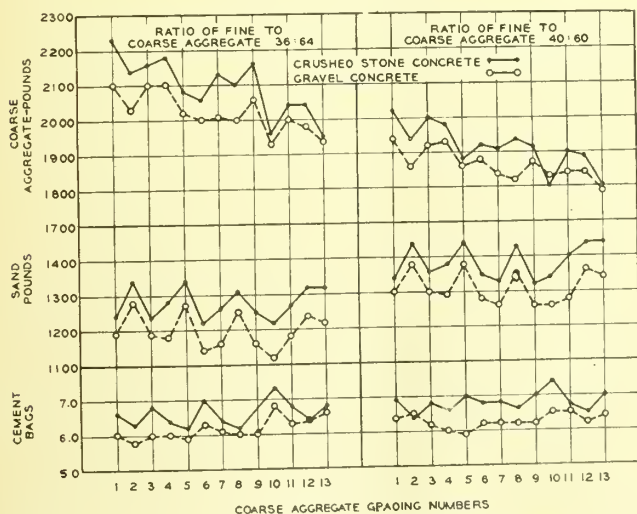


FIG. 9.—QUANTITIES OF MATERIALS REQUIRED FOR 1 CUBIC YARD OF CONCRETE

TABLE 13.—Quantities of material required for 1 cubic yard of concrete, using the proportions 1:1 3/4:3 1/2 by volume (nominal mix, series A)

Coarse aggregate grading No.	Stone			Gravel		
	Cement	Sand	Stone	Cement	Sand	Gravel
	Bags	Pounds	Pounds	Bags	Pounds	Pounds
1.....	6.1	1,160	2,350	5.9	1,120	2,230
2.....	6.4	1,210	2,180	6.2	1,190	2,120
3.....	6.3	1,190	2,330	5.9	1,120	2,210
4.....	6.3	1,200	2,290	5.7	1,080	2,160
5.....	6.8	1,280	2,230	6.3	1,200	2,140
6.....	6.2	1,170	2,220	5.7	1,090	2,140
7.....	6.2	1,180	2,250	5.8	1,100	2,140
8.....	6.5	1,230	2,220	6.1	1,160	2,080
9.....	6.2	1,170	2,250	5.8	1,100	2,180
10.....	6.3	1,200	2,160	5.8	1,100	2,140
11.....	6.4	1,220	2,210	5.9	1,110	2,130
12.....	6.6	1,250	2,170	6.2	1,170	2,110
13.....	6.6	1,260	2,090	6.1	1,150	2,060
Average.....	6.4	1,210	2,230	6.0	1,130	2,140

TABLE 14.—Solid volumes of material for 1 cubic yard of concrete for each mix of series A and total solids or density of concrete expressed as a percentage of maximum possible density

Coarse aggregate grading No.	Stone				Gravel			
	Solid volumes			Total solids	Solid volumes			Total solids
	Cement	Sand	Stone		Cement	Sand	Gravel	
	Cu. ft.	Cu. ft.	Cu. ft.	Per cent	Cu. ft.	Cu. ft.	Cu. ft.	Per cent
1.....	2.96	7.04	12.70	84	2.86	6.79	13.52	86
2.....	3.11	7.33	11.78	81	3.01	7.21	12.84	85
3.....	3.06	7.22	12.60	85	2.86	6.79	13.40	85
4.....	3.06	7.27	12.38	84	2.77	6.54	13.10	83
5.....	3.30	7.76	12.05	86	3.06	7.27	12.98	86
6.....	3.01	7.09	12.00	82	2.77	6.61	12.98	83
7.....	3.01	7.15	12.17	83	2.82	6.67	12.60	84
8.....	3.16	7.45	12.00	84	2.96	7.03	13.21	84
9.....	3.01	7.09	12.17	83	2.82	6.67	12.98	83
10.....	3.06	7.27	11.68	81	2.82	6.67	12.91	83
11.....	3.11	7.39	11.94	83	2.86	6.73	12.79	83
12.....	3.21	7.58	11.72	83	3.01	7.09	12.79	83
13.....	3.21	7.64	11.30	82	2.96	6.97	12.49	83

errors. For this reason, comparisons of the effect of aggregate type on yield should only be made on averages for all gradations, as in this way the number of specimens tend to largely eliminate the effect of experimental errors. The results for the individual gradings, however, are of interest to the extent that they indicate general relationships. For example, Figures 8 and 9 show at a glance the effect of gradings 2, 5, 8, 12, and 13 on the various quantities required to produce a cubic yard of concrete. The amount of cement and sand required is larger and the amount of coarse aggregate required is smaller than where the coarse aggregate contains the smaller sizes.

To bring out certain possible cost relations and illustrate the economic features involved in this study in a more graphic way than could be done by a comparison of quantities only, certain unit costs of materials have been assumed. These costs are said to apply fairly well to conditions in New Jersey and are as follows: Cement, 60 cents per bag; sand, 50 cents per ton; and coarse aggregate \$1.40 per ton. The resulting material costs per cubic yard are shown in Figures 10 and 11. Note the relative high cost resulting from the use of gradings 2, 5, 8, and 12 of series A. The value for grading No. 2 crushed stone is evidently in error as this point should be considerably higher than shown. On the basis of the above unit prices, the average cost of materials for 1 cubic yard of gravel concrete 1:1¾:3½ mix by volume is \$5.38, while the corresponding value for the crushed stone concrete is \$5.70, an increase of 32 cents, or approximately 6 per cent. These values and those for series B, together with the corresponding moduli of rupture are given in Table 17. Values may be calculated in a similar way for any combination of unit prices.

YIELD OF SERIES B

Data for quantities of material used and densities of concrete as determined for series B are shown in Tables 15 and 16. The quantities and corresponding costs are plotted in Figures, 8, 9, 10, and 11 and Table 17. These data show approximately the same relative difference in cost between the gravel and the stone concrete as in series A. It is interesting to note, however, that one factor which produces high costs in concrete proportioned as in series A, has just the opposite effect when proportioning is by the water-cement ratio trial method. When a contractor is working under a straight water-cement ratio specification, and sand is cheaper than coarse aggregate, as in this case, it is to his advantage to use a poorly graded stone or gravel because in so doing he is able to crowd more aggregate into his cement paste before passing the bounds of workability. On the other hand, the high cost of using a grading such as No. 10 with a large proportion of relatively small sizes is at once apparent, especially with the smaller proportions of sand.

SLABS OF EQUIVALENT STRENGTH DETERMINED

It is of interest to compute the relative theoretical thicknesses of slab required to produce pavements of equivalent transverse strength, using the average values of modulus of rupture for the two classes of concrete as shown by the tests of series A. For this purpose the so-called corner formula,² which is used

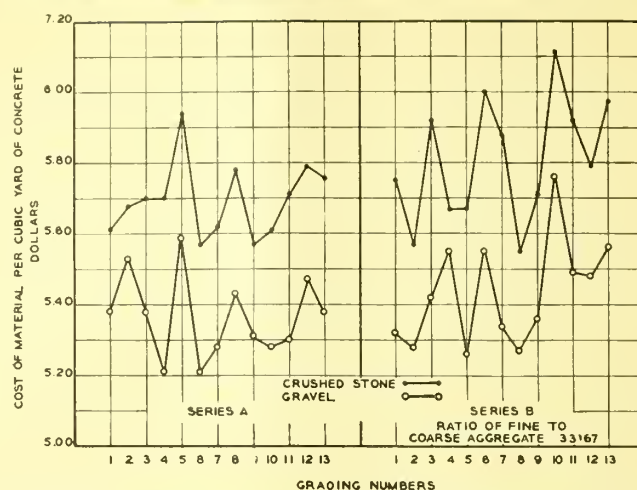


FIG. 10.—RELATION BETWEEN GRADING OF COARSE AGGREGATE AND TOTAL COST OF MATERIAL TO PRODUCE 1 CUBIC YARD OF CONCRETE

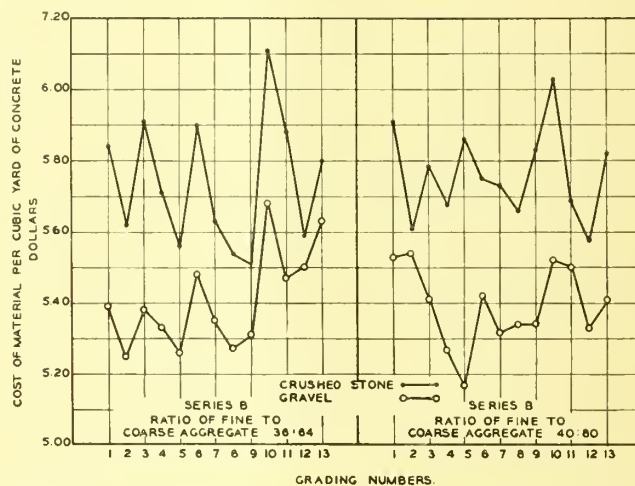


FIG. 11.—RELATION BETWEEN GRADING OF COARSE AGGREGATE AND TOTAL COST OF MATERIAL TO PRODUCE 1 CUBIC YARD OF CONCRETE

extensively in the calculation of edge thickness for concrete pavement slabs, is employed. This formula is usually expressed as follows:

$$d = \sqrt{\frac{3P}{s}}$$

in which d = depth of slab in inches;

s = allowable unit flexural stress in bending in the concrete;

and P = allowed wheel-load at corner in pounds.

In both cases the usual maximum allowable unit stress of one-half the modulus of rupture will be employed in the calculation. In the case of the gravel concrete this gives an allowable stress of 253 pounds per square inch and for the stone concrete a corresponding value of 285 pounds per square inch. Assuming a load at the corner of 8,000 pounds in each case, the value of d for the crushed-stone concrete reduces to 9.18 inches and for the gravel concrete to 9.74 inches, an increase of 0.56 inch, or 6.1 per cent. For equivalent slab strengths and on the basis of the flexural-strength values in the concrete obtained from these tests, pavements constructed of gravel concrete should be approximately one-half inch thicker than pavements constructed of crushed-stone concrete.

² OLDER, CLIFFORD. HIGHWAY RESEARCH IN ILLINOIS. Proc. A. S. C. E., 1924, p. 1180.

Reverting again to the previously assumed unit costs and adding 6.1 per cent to the cost of 1 cubic yard of gravel concrete as shown at \$5.38, a value of \$5.71 is obtained, which is almost exactly equal to the unit cost per cubic yard of the stone concrete. In other words, for the unit prices assumed, the cost of materials required to produce concrete pavements of the same slab strength is almost exactly the same for both aggregates. A similar cost analysis may, of course, be made with any other combination of unit prices. For example, assuming cement at 50 cents per bag, sand and gravel at \$1.20 per ton, and crushed stone at \$1.40 per ton, the unit cost of a cubic yard of gravel concrete (series A) is found to be \$4.96, with \$5.49 as the corresponding cost for the stone concrete, an increase of 53 cents. In this case, adding 6½ per cent to the cost of the gravel concrete to care for the added thickness, makes the total cost of material in a gravel concrete slab of equivalent strength \$5.26 as against \$5.49 for the stone concrete.

TABLE 15.—Quantities of materials required for 1 cubic yard of concrete, series B

RATIO OF FINE TO COARSE AGGREGATE 33 : 67

Coarse aggregate grading No.	Stone			Gravel		
	Cement	Sand	Stone	Cement	Sand	Gravel
	Bags	Pounds	Pounds	Bags	Pounds	Pounds
1.....	6.4	1,120	2,320	5.9	1,080	2,180
2.....	6.1	1,260	2,300	5.7	1,210	2,200
3.....	6.7	1,150	2,300	6.0	1,080	2,170
4.....	6.3	1,190	2,300	6.2	1,080	2,200
5.....	6.3	1,260	2,230	5.8	1,170	2,120
6.....	7.0	1,130	2,170	6.4	1,050	2,100
7.....	6.8	1,130	2,190	6.0	1,080	2,130
8.....	6.2	1,210	2,210	5.8	1,160	2,120
9.....	6.4	1,140	2,250	5.9	1,080	2,180
10.....	7.3	1,140	2,080	6.7	1,050	2,080
11.....	6.9	1,170	2,150	6.2	1,080	2,100
12.....	6.7	1,210	2,140	6.1	1,180	2,170
13.....	7.0	1,220	2,060	6.3	1,130	2,040
Average.....	6.6	1,180	2,210	6.1	1,110	2,140

RATIO OF FINE TO COARSE AGGREGATE 36 : 64

1.....	6.6	1,240	2,230	6.0	1,190	2,100
2.....	6.3	1,340	2,140	5.8	1,280	2,030
3.....	6.8	1,240	2,160	6.0	1,190	2,100
4.....	6.4	1,280	2,180	6.0	1,180	2,100
5.....	6.2	1,340	2,080	5.9	1,270	2,020
6.....	7.0	1,220	2,060	6.3	1,140	2,000
7.....	6.4	1,260	2,130	6.1	1,160	2,010
8.....	6.2	1,310	2,100	6.0	1,250	2,000
9.....	6.1	1,250	2,160	6.0	1,160	2,060
10.....	7.4	1,220	1,960	6.8	1,120	1,930
11.....	6.8	1,270	2,040	6.3	1,180	2,000
12.....	6.4	1,320	2,040	6.4	1,240	1,980
13.....	6.8	1,320	1,950	6.6	1,220	1,940
Average.....	6.6	1,280	2,100	6.2	1,200	2,020

RATIO OF FINE TO COARSE AGGREGATE 40 : 60

1.....	6.9	1,340	2,020	6.4	1,300	1,940
2.....	6.4	1,440	1,940	6.5	1,380	1,860
3.....	6.8	1,360	2,000	6.2	1,300	1,920
4.....	6.6	1,380	1,980	6.0	1,290	1,930
5.....	7.0	1,440	1,880	5.9	1,380	1,860
6.....	6.8	1,350	1,920	6.2	1,280	1,880
7.....	6.8	1,330	1,910	6.2	1,260	1,840
8.....	6.6	1,430	1,940	6.2	1,350	1,820
9.....	7.0	1,320	1,920	6.2	1,260	1,870
10.....	7.4	1,340	1,800	6.5	1,250	1,830
11.....	6.7	1,400	1,900	6.5	1,280	1,840
12.....	6.5	1,440	1,890	6.2	1,360	1,840
13.....	7.0	1,440	1,800	6.4	1,340	1,790
Average.....	6.8	1,390	1,920	6.3	1,310	1,860

These figures are given solely to illustrate how fluctuations in material prices affect relative costs and not with any idea that they may be applied to any specific conditions. It should also be noted that, theoretically, there will be a somewhat higher labor cost in placing the gravel due to its extra thickness, although this item might possibly be somewhat neutralized by the greater workability of the gravel concrete as compared with the crushed-stone concrete.

TABLE 16.—Solid volumes of material for 1 cubic yard of concrete for each mix of series B and total solids or density of concrete expressed as a percentage of maximum possible density

RATIO OF FINE TO COARSE AGGREGATE 33 : 67

Coarse aggregate grading No.	Stone				Gravel			
	Solid volumes			Total solids	Solid volumes			Total solids
	Cement	Sand	Stone		Cement	Sand	Gravel	
1.....	Cu. ft.	Cu. ft.	Cu. ft.	Per ct.	Cu. ft.	Cu. ft.	Cu. ft.	Per ct.
1.....	3.11	6.79	12.54	83	2.86	6.54	13.21	84
2.....	2.97	7.64	12.42	85	2.77	7.34	13.34	87
3.....	3.26	6.97	12.42	84	2.91	6.54	13.35	84
4.....	3.08	7.22	12.42	84	3.01	6.54	13.34	85
5.....	3.06	7.64	12.05	84	2.81	7.10	12.85	84
6.....	3.40	6.85	11.74	82	3.11	6.36	12.71	83
7.....	3.30	6.85	11.84	81	2.91	6.54	12.91	84
8.....	3.01	7.33	11.95	83	2.81	7.03	12.85	84
9.....	3.11	6.97	12.16	82	2.86	6.54	13.20	84
10.....	3.54	6.91	11.25	80	3.25	6.36	12.60	82
11.....	3.35	7.09	11.62	82	3.01	6.54	12.71	82
12.....	3.25	7.34	11.56	82	2.96	7.15	13.16	86
13.....	3.40	7.40	11.13	81	3.05	6.84	12.36	82

RATIO OF FINE TO COARSE AGGREGATE 36 : 64

1.....	3.21	7.52	12.06	84	2.91	7.22	12.73	85
2.....	3.06	8.12	11.57	84	2.82	7.76	12.30	85
3.....	3.30	7.52	11.67	83	2.91	7.72	12.73	85
4.....	3.11	7.76	11.79	84	2.91	7.16	12.23	85
5.....	3.01	8.12	11.24	83	2.86	7.70	12.24	84
6.....	3.40	7.40	11.14	81	3.06	6.91	12.12	82
7.....	3.11	7.64	11.51	82	2.96	7.03	12.18	82
8.....	3.01	7.94	11.36	83	2.91	7.58	12.12	84
9.....	2.96	7.58	11.68	82	2.91	6.91	12.48	83
10.....	3.60	7.40	10.60	80	3.30	6.79	11.70	81
11.....	3.30	7.70	11.03	82	3.01	7.15	12.12	83
12.....	3.11	8.00	11.03	82	3.11	7.52	12.00	84
13.....	3.30	8.00	10.54	81	3.21	7.39	11.75	83

RATIO OF FINE TO COARSE AGGREGATE 40 : 60

1.....	3.35	8.12	10.91	83	3.11	7.88	11.75	84
2.....	3.11	8.73	10.49	83	3.16	8.36	11.27	84
3.....	3.30	8.24	10.81	83	3.01	7.88	11.64	83
4.....	3.20	8.36	10.70	82	2.91	7.82	11.69	83
5.....	3.40	8.73	10.16	83	2.86	8.36	11.27	83
6.....	3.30	8.18	10.38	81	3.01	7.76	11.39	82
7.....	3.30	8.06	10.32	80	3.01	7.64	11.15	81
8.....	3.20	8.67	10.49	83	3.01	8.18	11.03	82
9.....	3.40	8.00	10.38	81	3.01	7.64	11.33	81
10.....	3.59	8.12	9.73	79	3.16	7.64	11.09	81
11.....	3.26	8.49	10.27	82	3.16	7.76	11.15	82
12.....	3.16	8.73	10.21	82	3.01	8.24	11.15	83
13.....	3.40	8.73	9.73	81	3.11	8.12	10.85	82

TABLE 17.—Average costs per cubic yard of concrete, series A and B

Series	Coarse aggregate	Ratio of fine to coarse aggregate	Cost of concrete per cubic yard	Average modulus of rupture 28 days
A	Stone.....	33:67	\$5.70	570
A	Gravel.....	33:67	5.38	505
B	Stone.....	33:67	5.80	586
B	Gravel.....	33:67	5.45	525
	Stone.....	36:64	5.75	590
	Gravel.....	36:64	5.40	525
	Stone.....	40:60	5.75	580
	Gravel.....	40:60	5.40	515

Costs based on the following assumed unit prices: Cement, \$0.60 per bag; sand, \$0.50 per ton; coarse aggregate, \$1.40 per ton.

CONCLUSIONS PRESENTED

It is not intended that the conclusions given below and which are based on the results of these tests shall be considered as applicable to crushed trap rock and gravel of a different type and quality than those employed in this investigation, or to kindred aggregates produced and marketed under conditions differing from those used, and should not be interpreted as applying to crushed-stone or gravel aggregates in general. These conclusions are as follows:

1. That when coarse aggregates comparable in quality to those used in these tests are employed in the construction of concrete pavements in New Jersey under existing specifications:

(a) Concrete in which crushed trap rock is used as coarse aggregate will average about 12 per cent higher in flexural strength than concrete in which gravel is used as coarse aggregate.

(b) There will be practically no difference in the crushing strength of crushed trap-rock concrete and gravel concrete.

(c) There will be practically no difference in the absorption of crushed trap-rock concrete and gravel concrete.

(d) For equivalent flexural slab strengths, a pavement constructed of gravel concrete should have a depth approximately one-half inch greater than a pavement constructed of crushed trap-rock concrete.

(e) The cost of the materials required for a unit volume of crushed trap-rock concrete will as a rule be greater than the cost of materials required for an equivalent volume of gravel concrete.

2. That when coarse aggregates comparable to those used in these tests are used in concrete mixtures designed for a given strength by the so-called water-cement ratio trial method:

(a) The flexural strength of the crushed trap-rock concrete will average about 11 per cent higher than the gravel concrete.

(b) There will be practically no difference in the crushing strength of the crushed trap-rock concrete and the gravel concrete.

In addition to the above, the following indications as to effect of gradation on strength and yield when the concrete is proportioned by fixed volume as well as by the water-cement ratio theory may be stated:

(1) That the gradation of the coarse aggregate has very little direct effect upon the strength of the concrete.

(2) That when proportioned by the water-cement ratio trial method, variations in the fine-coarse aggregate ratio of from 1:2 to 2:3 do not affect the strength of the concrete for a given sand and for a given water-cement ratio.

(3) That variation in coarse aggregate grading will greatly affect the yield of concrete and therefore its cost, when the concrete is proportioned either in the usual way or by the water-cement ratio method.

(4) That the use of well-graded coarse aggregate will increase the yield when proportioned by the usual method, but exactly the reverse is the case when the concrete is proportioned by the water-cement ratio method.

(Continued from page 262)

but it is one which is rather hard on the profits which might otherwise be had from the job.

Table 8 shows a number of typical readings of the time required to move both crawler and wheel-traction type shovels under fair to good field conditions. The average time required for moving the wheel-traction type operating on mats was more than five times that required for moving the improved crawler type. In shallow cuts where much moving is required the shovel equipped with wheel traction operating on mats is under a severe handicap. This is also true for some of the older crawler types which normally require an additional man when moving. In general, and under fair field conditions, it should be possible to keep the average time per move for a $\frac{3}{4}$ -yard crawler type shovel of the better type within 15 seconds and within 75 seconds for a similar shovel equipped with wheel traction and operating on mats. If the average time per move approaches 30 seconds under ordinary field conditions for the crawler shovel or 150 seconds for the wheel shovel, we may safely conclude that either the operator is slow or the mechanism needs adjustment or repair. For blocking or leveling the shovel on steep grades at least two wide wedge-shaped blocks reinforced with strap iron and light bolts should be provided. Such blocks can readily be handled by the pitman and will pay for themselves in a few hours where conditions are such that blocking is required.

SHOVEL OPERATOR A MOST IMPORTANT FACTOR

A high degree of efficiency in power-shovel operation can only be secured through the proper coordination of

TABLE 8.—Shovel movements classified according to time required

Time required (seconds)	Movements of crawler- traction shovels	Movements of wheel- traction shovels operating on mats	Time required (seconds)	Movements of crawler- traction shovels	Movements of wheel- traction shovels operating on mats
	Number	Number		Number	Number
8 to 10.....	6		50 to 60.....	2	5
10 to 12.....	29		60 to 70.....	1	12
12 to 14.....	32		70 to 80.....		28
14 to 16.....	20		80 to 90.....	2	7
16 to 18.....	14		90 to 100.....		10
18 to 20.....	8		100 to 125.....		16
20 to 25.....	23		125 to 150.....		7
25 to 30.....	10		150 to 175.....		4
30 to 35.....	7		175 to 200.....		1
35 to 40.....	8		Above 200.....		6
40 to 50.....	6	3			

several factors. The first and most apparent is the operator. The ideal operator is a man gifted with quick reaction, a true eye, good judgment, great endurance, and a high degree of skill and experience. He should know the possibilities as well as the limitations of the shovel and be able to maintain it in first-class condition.

But it is not enough simply to secure a good operator. Except where casting is involved, the operator can dig no more material than the available equipment can haul, and he can only dig when hauling units are actually in place for loading. If the supply of wagons or trucks is inadequate to handle full shovel production, or if their operation is such as to interfere with the steady, methodical operation of the shovel, the fault lies with the management.

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Report of the Chief of the Bureau of Public Roads, 1925.
Report of the Chief of the Bureau of Public Roads, 1927.

DEPARTMENT BULLETINS

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- 257D. Progress Report of Experiments in Dust Prevention and Road Preservation, 1914.
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- Vol. 5, No. 19, D- 3. Relation Between Properties of Hardness and Toughness of Road-Building Rock.
- Vol. 5, No. 24, D- 6. A New Penetration Needle for Use in Testing Bituminous Materials.
- Vol. 6, No. 6, D- 8. Tests of Three Large-Sized Reinforced-Concrete Slabs Under Concentrated Loading.
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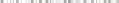
* Department supply exhausted.

UNITED STATES DEPARTMENT OF AGRICULTURE
BUREAU OF PUBLIC ROADS
STATUS OF FEDERAL AID HIGHWAY CONSTRUCTION

AS OF
JANUARY 31, 1928

STATES	FISCAL YEARS 1917-1927					FISCAL YEAR 1928					BALANCE OF FEDERAL AID FUND AVAILABLE FOR NEW PROJECTS		
	PROJECTS COMPLETED PRIOR TO JULY 1, 1927					* PROJECTS UNDER CONSTRUCTION					PROJECTS APPROVED FOR CONSTRUCTION		
	PROJECTS COMPLETED SINCE JUNE 30, 1927												
	TOTAL COST	FEDERAL AID	MILES	TOTAL COST	FEDERAL AID	MILES	ESTIMATED COST	FEDERAL AID ALLOTTED	MILES	ESTIMATED COST	FEDERAL AID ALLOTTED	MILES	
Alabama	\$ 20,061,371.68	\$ 9,615,099.94	1,400.2	\$ 725,370.17	\$ 354,700.69	42.1	\$ 9,754,097.56	\$ 4,254,934.60	510.2	\$ 630,145.64	\$ 315,072.81	19.9	2,904,349.96
Arizona	11,803,560.70	6,447,183.27	800.8	624,598.32	451,055.92	15.7	1,087,792.63	827,674.26	66.1	827,674.26	17,264.44	0.2	3,990,160.11
Arkansas	22,337,014.63	5,555,192.75	1,560.6	52,245.29	26,122.64	0.1	4,740,352.25	2,115,027.41	241.0	34,528.89			2,499,142.20
California	35,129,269.04	15,967,026.82	1,306.3	3,686,309.77	1,699,840.46	84.7	7,617,482.96	3,428,764.64	154.2	230,101.00	134,662.03	9.4	4,812,463.15
Colorado	15,497,121.91	7,934,298.91	829.0	91,335.74	48,486.64	0.9	7,076,482.68	3,468,895.51	285.1	35,740.10	20,057.34	2.9	3,613,994.50
Connecticut	6,397,392.29	2,444,000.94	137.3	1,352,957.40	367,625.33	17.7	5,712,748.89	1,597,373.38	69.6	1,139,177.58	191,246.72	12.0	680,332.03
Delaware	6,237,026.55	2,345,572.42	159.5	486,740.02	232,041.85	29.4	934,367.77	262,068.47	16.3				365,625.26
Florida	7,476,956.31	3,627,912.60	245.1	4,058,593.22	1,947,518.78	75.4	5,750,741.38	2,386,757.80	133.0	1,363,436.03	562,525.30	37.8	1,361,001.52
Georgia	31,951,436.50	15,101,232.40	2,173.6	5,271,289.20	2,984,490.77	225.5	4,512,644.56	2,241,128.26	167.4	454,728.07	227,354.02	34.8	1,835,981.55
Idaho	13,225,515.45	7,075,527.16	835.5	1,514,248.94	949,594.29	111.0	16,258,991.84	1,306,353.28	142.3	116,000.00	65,500.00	10.8	1,030,807.27
Illinois	48,538,982.16	22,741,516.60	1,530.8	408,484.92	191,977.92	15.0	16,284,820.84	7,591,175.60	538.6	2,136,204.62	1,059,102.31	76.3	4,398,079.57
Indiana	23,372,717.74	11,238,569.20	732.5	997,394.00	482,777.67	34.5	16,934,990.86	8,108,138.77	513.2	2,249,564.67	1,017,689.96	80.1	1,205,415.80
Iowa	34,306,139.86	14,395,603.75	2,484.4	5,461,954.45	2,558,101.78	334.0	10,537,012.24	4,643,249.80	283.1	2,982,871.37	1,033,579.56	17.4	935,318.01
Kansas	37,442,051.61	14,730,829.48	1,495.2	4,065,705.11	1,797,079.21	251.8	12,302,702.10	4,824,488.01	639.4	16,479.77	62,000.00	6.6	2,180,742.30
Kentucky	23,215,500.53	9,510,634.75	874.9	1,125,250.03	513,559.73	53.5	9,729,207.67	4,602,263.41	418.9	50,000.00	25,000.00	5.4	1,400,257.11
Louisiana	15,877,552.20	7,093,892.21	1,178.7	1,332,121.58	597,801.48	49.7	3,950,319.64	1,822,681.80	139.4	2,259,082.75	1,111,115.29	83.3	679,506.22
Maine	10,584,800.06	4,858,452.67	357.6	1,606,901.19	594,650.99	51.9	1,632,359.15	594,670.62	43.9	87,912.01	43,956.00	5.6	1,725,312.82
Maryland	11,730,203.93	5,524,938.27	477.8	774,726.27	354,681.06	42.3	1,441,899.31	696,347.17	62.2				619,115.50
Massachusetts	20,570,246.02	7,425,928.15	410.4	773,729.83	159,264.74	9.6	6,742,463.44	1,865,290.01	115.4	523,071.01	201,480.00	13.4	2,635,671.10
Michigan	31,977,248.37	14,328,484.99	1,084.2	3,276,113.43	1,507,222.57	118.5	13,204,609.57	5,865,815.50	347.6	1,282,954.10	485,027.00	32.3	2,580,101.94
Minnesota	45,099,648.47	19,045,145.67	3,643.5	5,952,424.86	2,001,398.30	248.7	3,948,281.25	1,133,100.00	216.5	1,759,628.39	408,000.00	50.8	1,235,471.43
Mississippi	19,331,230.75	9,004,294.62	1,314.1	1,962,975.53	952,031.17	110.6	5,932,705.57	2,865,019.70	303.7	397,399.14	143,699.05	29.9	1,733,592.45
Missouri	42,359,230.41	19,691,036.48	1,944.8	3,675,293.66	1,601,051.53	124.1	7,237,328.27	3,614,252.43	336.0	595,687.27	132,843.82	24.4	2,922,770.84
Montana	12,854,995.72	7,297,268.69	1,151.6	4,421,710.47	310,860.44	51.2	4,127,080.59	2,803,091.44	334.0	1,017,860.58	553,621.91	114.3	5,554,159.02
Nebraska	16,157,040.25	7,759,365.39	2,246.6	4,294,321.03	2,068,912.12	412.9	11,765,933.85	5,804,015.44	1167.1	490,467.82	242,845.23	48.5	1,960,194.82
Nevada	13,335,250.94	7,593,988.06	853.6	576,866.41	495,568.11	67.8	1,723,110.43	1,504,094.33	186.1	56,261.35	49,352.46	6.7	1,063,535.42
New Hampshire	5,559,897.76	2,778,328.05	284.8	694,506.95	315,165.24	23.7	930,323.53	410,717.07	26.1	60,672.13	23,295.00	1.8	372,536.64
New Jersey	22,229,240.08	7,495,354.48	316.3	3,953,328.30	1,012,035.00	67.5	3,910,282.53	817,294.17	53.3	236,240.90	77,337.35	5.6	935,122.00
New Mexico	13,335,250.94	7,593,988.06	1,505.2	810,649.68	535,454.33	67.7	3,182,754.13	2,448,505.46	207.5	4,636,600.00	909,952.50	58.2	2,448,852.15
New York	54,193,095.44	21,693,955.65	1,439.3	4,359,109.53	1,422,854.62	95.7	43,416,936.00	10,565,541.45	656.7				5,716,295.78
North Carolina	35,295,949.21	14,518,993.16	1,450.1	1,569,316.80	729,783.35	63.3	3,296,998.16	1,577,515.52	92.9	151,250.00	74,000.00	6.4	2,245,269.97
North Dakota	15,891,558.55	7,746,293.68	2,715.6	2,949,130.42	1,674,127.76	369.1	4,342,269.12	2,141,697.39	671.4	525,413.30	238,017.11	101.2	1,335,914.06
Ohio	52,621,391.49	19,331,376.76	1,151.0	4,511,431.24	1,978,559.59	156.0	10,200,058.00	3,886,995.03	258.2	2,560,392.00	1,053,795.00	61.2	5,001,209.62
Oklahoma	30,331,957.08	14,117,589.21	1,269.1	789,742.25	379,754.99	9.6	6,480,678.84	2,924,580.49	396.5	1,044,745.17	501,291.34	67.5	1,637,522.06
Oregon	19,593,598.76	10,041,452.94	1,055.0	251,199.81	137,760.38	111.3	2,711,328.63	1,456,357.68	75.6	121,356.95	58,953.27	0.9	1,586,711.23
Pennsylvania	77,776,174.22	26,317,620.32	1,534.3	4,413,200.23	1,393,613.17	97.8	17,992,594.54	5,391,479.08	337.3	1,855,518.07	545,837.47	36.0	4,359,749.96
Rhode Island	5,233,413.38	1,998,479.08	116.0	700,482.52	227,205.00	15.1	1,448,390.20	389,422.41	23.4	357,495.45	99,990.00	6.6	683,722.53
South Carolina	17,002,039.93	7,556,998.90	1,568.4	2,124,613.35	1,013,294.39	69.0	8,297,698.91	2,319,625.72	227.7	1,238,736.87	137,000.00	18.0	919,166.09
South Dakota	19,252,083.24	9,507,525.54	2,502.9	497,551.21	264,835.59	98.5	4,872,560.95	2,578,121.58	715.6	512,398.46	219,281.90	92.7	1,041,069.39
Tennessee	24,283,035.03	11,551,457.65	888.7	2,614,430.80	1,167,735.69	65.4	8,018,992.93	3,363,211.14	215.6	1,404,201.48	691,951.06	62.3	1,733,013.36
Texas	78,130,246.37	31,565,980.45	5,465.4	7,991,555.08	3,651,419.03	274.5	13,971,441.56	6,036,203.24	467.5	4,031,493.80	1,532,371.58	120.4	6,819,319.70
Utah	9,154,377.33	3,567,079.95	628.9	854,916.56	628,347.38	80.5	2,735,358.04	2,035,223.37	169.9	224,936.28	34,628.72	4.0	1,048,498.58
Vermont	5,037,118.23	2,348,956.01	152.7	1,442,701.40	571,922.67	27.9	1,939,714.80	683,331.78	43.7	506,966.19	192,019.27	6.6	395,746.54
Virginia	26,844,025.24	12,537,143.25	1,189.9	7,026,180.54	447,174.68	27.3	7,496,157.15	1,930,082.88	103.0				1,275,355.92
Washington	18,184,505.97	8,246,551.95	711.4	1,052,488.88	475,735.32	22.3	4,559,312.97	1,983,270.53	123.3	50,548.85	11,329.47	0.1	1,702,645.73
West Virginia	10,424,847.32	4,573,748.01	419.1	1,919,689.39	774,875.18	51.4	6,407,489.46	2,691,820.10	228.9	129,443.32	64,121.66	6.7	844,770.05
Wisconsin	27,891,502.16	11,847,859.90	1,229.5	4,462,335.78	2,159,487.04	206.5	8,060,777.68	3,493,549.85	263.1				3,673,586.21
Wyoming	12,650,712.15	7,139,267.05	1,315.9	1,503,995.96	962,836.11	83.6	2,169,196.59	1,382,696.58	224.1				955,390.26
Hawaii	343,584.15	97,440.00	6.5	160,036.91	70,440.00	4.7	1,624,080.08	491,922.64	25.0				1,171,600.38
TOTALS	1,154,740,501.48	510,007,691.24	80,957.6	105,952,266.53	47,252,452.60	4,567.0	341,146,578.97	140,815,834.61	12,950.4	39,288,931.01	14,638,780.36	1,420.3	104,900,241.19
													4,470.0

* Includes projects reported completed (final vouchers not yet paid) totaling: Estimated cost \$ 116,807,474.19 Federal aid \$ 49,285,345.71 Miles 4,470.0



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